ERDC/ITL TR-03-1

Information Technology Laboratory

US Army Corps of Engineers®
Engineer Research and Development Center

Innovations for Navigation Projects Research Program

Investigative Study for Underwater Construction of Lock Floors and Culverts

Barry D. Fehl, Thurman D. Gaddie, and Kevin Abraham

January 2003

Innovations for Navigation
Projects Research Program

ERDC/ITL TR-03-1
January 2003

Investigative Study for Underwater Construction of Lock Floors and Culverts

by Barry D. Fehl

GEI Consultants, Inc.
243 N. Lindbergh Blvd.
Suite 312
St. Louis, MO 63141-7851

Thurman W. Gaddie

Consultant
151 Miami Lakes Drive
Milford, OH 45150-5809

Kevin Abraham

Information Technology Laboratory
U.S. Army Engineer Research and Development Center
3909 Halls Ferry Road
Vicksburg, MS 39180-6199

Final report

Prepared for U.S. Army Corps of Engineers
Washington, DC 20314-1000

Under INP Work Unit 33147

Contents

List of Figures

Preface

The work described in this report was authorized by Headquarters, U.S. Army Corps of Engineers (HQUSACE), as part of the Innovations for Navigation Projects (INP) Research Program. The work was performed under the Work Unit 33147, "Lock Floor Slab and Culvert Construction Underwater."

Dr. Tony C. Liu was the INP Coordinator at the Directorate of Research and Development, HQUSACE. Program Monitors were Mr. Mike Kidby and Ms. Anjana Chudgar, HQUSACE. Mr. William H. McAnally of the U.S. Army Engineer Research and Development Center (ERDC) Coastal and Hydraulics Laboratory was the Lead Technical Director for Navigation Systems; Dr. Stanley C. Woodson, ERDC Geotechnical and Structures Laboratory (GSL), was the INP Program Manager; and Mr. Kevin Abraham, ERDC Information Technology Laboratory (ITL), was the Principal Investigator.

This report was prepared by Dr. Barry D. Fehl, GEI Consultants, Inc., St. Louis, MO, Mr. Thurman W. Gaddie, Consultant, Milford, OH, and Mr. Kevin Abraham, ITL, under the supervision of Mr. H. Wayne Jones, Chief, Computer-Aided Engineering Division; Dr. Jeffery P. Holland, Director, ITL; and Dr. David W. Pittman, Acting Director, GSL.

At the time of publication of this report, Dr. James R. Houston was Director of ERDC; COL John W. Morris III, EN, was Commander and Executive Director.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
feet	0.3048	meters
inches	25.4	millimeters
pounds (mass)	0.45359237	kilograms
pounds (force) per square inch	6.894757	kilopascals

1 Introduction

Construction of locks by the U.S. Army Corps of Engineers has always been performed in the dry. Several means have been used to accomplish this. Most often, a cofferdam was built in the river, and pumps were used to dewater the cofferdam. In some cases a new channel was constructed with earthen dikes at each end of the channel and, once the lock was complete, the dikes were removed. In other cases, an existing slough or old channel was used, and then the river was diverted into this channel.

These methods served the Corps of Engineers quite well for a number of years. Preparation of foundations could be done with conventional earth-moving equipment; formwork was constructed by carpenters and placed precisely using surveying methods; reinforcing was placed carefully using tape measures; and concrete was placed from an onsite batch plant and vibrated in place to ensure elimination of voids. Well-established methods for quality assurance and quality control were used to ensure that the lock was constructed properly, and the result was lock structures all over the United States that continued to serve the public adequately despite being in excess of 60 years old in some cases.

It is now becoming necessary to replace some of these aging structures, in some cases because of their age, but in many other cases because the capacity is insufficient to meet the barge industry's needs. However, the need to reduce the costs of constructing locks and dams on the Nation's waterways has also become necessary. The funding is not available to permit the construction of all of the new and lock extension projects in the time frame that the barge industry feels is necessary. Therefore, it is important to discover less expensive approaches to lock construction.

In an effort to accomplish construction of new locks and the extension of existing locks, the Corps has investigated floating and lifting structures into place and building projects in the wet. Cofferdams by themselves are often 20 percent of the project cost; thus, by lifting or floating a structure into place or by constructing it in the wet, these costs can be avoided.

This report focuses on a couple of specific aspects of in-the-wet construction of locks—floor slabs and culverts. Various lock wall configurations and even entire U-frame lock sections have been investigated in a variety of studies since the Corps began exploring innovative methods for construction of locks began in 1992. While there has been some investigation into how to construct lock floors

and slabs as separate entities to the lock structure, these investigations have not been as detailed as those for lock walls.

The primary objective of this report is to provide a detailed investigation into the construction and design of lock floor slabs and culverts in an underwater condition. To accomplish this objective, plans must be established to identify possible construction methods and, also, those areas in which existing design parameters may need to be adjusted because of the underwater condition in which the structure will be built. The plans that are used for evaluating construction methods and design parameters must be plans that will have a reasonable expectation of being implemented.

An extensive review has been undertaken to ensure that plans selected for this report have a reasonable expectation that they can be advantageously constructed. This review included Design and Construction Innovations for Locks and Dams, Phase I, Feasibility (1993); Upper Mississippi River-Illinois Waterway System Study, Large-Scale Measures of Reducing Traffic Congestion, Conceptual Lock Designs (1996); Second Generation of Innovations, Innovations for Navigation Projects (INP) Research and Development Workshop (1997); Ohio River Main Stem Study, Prototype Alternatives (1998); and Upper Mississippi River-Illinois Waterway System Navigation Study: Interim Revised Lock Extension Design Concept (1999).

From this documentation, nine plans were selected and, to ensure that these were reasonable plans to consider, they were presented for review by members of the Field Advisory Committee (FAC) for the INP Research Program work unit "Lock Floor Slab and Culvert Construction Underwater." Discussions resulted in the elimination of three of the plans and inclusion of one new plan, as well as an alternative to one of the existing plans. Therefore, seven plans will be used in the investigation.

In addition to reviewing literature on possible innovative plans to be used in actual construction, a review was also made of possible construction methods and techniques that could be used for the underwater construction of slabs and culverts. This review included a literature search, an Internet search, discussions with designers and other researchers, and review of work performed within other work units of the INP Program. This included reviews of underwater concrete techniques, offshore pipe placement, underwater tunnel construction, and construction of articulated mats underwater.

After selecting the innovative plans and gathering information on underwater construction, investigation of these items could begin. The innovative plans were first evaluated against a set of criteria to establish the pros and cons of each plan and its applicability to underwater construction. These criteria included issues related to maintenance, longevity, projected ease or difficulty of implementation (i.e. constructibility), risks to the operation of the lock, cost, environmental concerns, hydraulic efficiency, construction time, and any limitations that might be associated with a given plan. This evaluation will permit designers to evaluate various options for constructing floor slabs and culverts with a better

understanding of the benefits and the restrictions associated with a given approach. These items are discussed in Chapter 2. A review of the studies done in the past and referenced above is also included, as well as discussions of other possible technologies that could have been used. Also discussed are the considerations that should be given to a project with respect to the foundation type and whether the construction is for a new lock or a lock extension.

As stated earlier, the selected plans will be used to evaluate design parameters with respect to changes in design parameters that are needed due to the construction being performed underwater. It must be understood that not every design parameter will need to be revised; therefore, the first step is to identify those parameters that may need to be revised. Subsequent to the publication of this report, studies will be performed to establish the proper values and procedures for those design parameters identified. As an introduction to identifying the design parameters in Chapter 3, an overview of existing parameters will be given, as well as a general discussion about the impact of in-the-wet construction on design parameters.

Chapter 4 focuses on construction methods and reviews construction methods that have been used for conventional in-the-dry construction of lock floor slabs and culverts. An overview of available in-the-wet construction technologies is presented, and this leads into discussion of which technologies can be used to construct the plans presented in Chapter 2. The discussion of in-the-wet technologies includes how these technologies can be applied to the proposed plans for constructing lock floor slabs and culverts underwater.

2 Overview of Plans

While the objective of this research is to identify construction techniques and methods and changes in design parameters for the underwater construction of lock floor slabs and culverts, the first step is to identify plans that are considered as reasonable candidates for future construction. While some construction techniques and methods could be identified for construction of these types of components without a specific plan, specific plans will allow more detailed investigations to be done. Likewise, specific plans will provide an opportunity to more thoroughly review design parameters and provide a more comprehensive identification of design parameters that may require changes for underwater construction.

Nine plans were originally selected for use in evaluating construction methods and identifying design parameters that might need to be revised. However, after a review of these plans by the Work Unit FAC, three plans were eliminated and one was added. This resulted in seven plans to be investigated, with one of the plans having an alternative wall scheme.

This chapter provides a description of each of the seven plans selected. Each plan is evaluated based on maintenance, longevity, projected ease or difficulty of implementation (i.e., constructibility), risks to operation of the lock, cost, environmental considerations, hydraulic efficiency (culverts only), construction time, and limitations. Discussion of each plan will address each of these items and identify the positive and negative aspects associated with the plan for the particular item. This should be beneficial to designers as they consider the options presented by these seven plans. In addition to the items listed above, the following sections include some discussion for each plan with respect to use of the plan for a lock extension or for a new lock, and how the plan can be used in a rock- or soil-founded condition.

During review of the seven plans it became apparent that, for the longevity and environmental factors, there was little to discuss. When considering the longevity of the plans, it became clear that there was little reason to expect that all would not perform well. For environmental considerations, one of the primary concerns is where precast units might be built and the associated impacts on the environment. For many of the float-in plans of walls, graving yards near the construction site are common. However, for floor slabs and culverts, the size of any precast unit is such that construction of these units would likely occur in existing precast yards. Another concern would be tremie concrete; however,

these environmental concerns would be the same regardless of which plan is used. In addition, experiments are currently being planned to ascertain the effects of the tremie process on the river environment.

Some discussion of construction techniques will be included in this chapter; however, more detailed discussion of possible construction techniques is presented in Chapter 4.

Plan 1

General

Plan 1 comes from the Upper Mississippi River-Illinois Waterway System (UMR-IWS) Navigation Study: Interim Revised Lock Extension Design Concept (1999). This report was developed subsequent to the original studies done on the UMR-IWS. The economic analysis from these studies did not show a significant economic benefit in extending locks or building new locks on the upper Mississippi River or on the Illinois Waterway. In an effort to further reduce construction costs, the study was extended to investigate other cost-saving approaches that might be taken in the extension of the 600-ft locks to 1,200 ft, with results reported in the above-referenced report.[1]

The primary thrust of this work was to use the existing downstream guide wall as the rubbing surface on the landward side of the lock extension and reinforce the guide wall with sheet-pile cells constructed behind it. On the riverward side of the extension, sheet-pile cells would also be constructed, and prefabricated rubbing surfaces would have to be attached to them. Deep well casings are to be included in construction of the sheet-pile cells to permit dewatering and to reduce structural requirements for the floor system. In addition, the sheet piles are driven 22 ft below the founding level to lengthen the seepage path from the river during dewatering.

An elevation of the sheet-pile cell to be used as an extension of the existing river wall is shown in Figure 1, and an elevation of the existing guide wall being reinforced by a sheet-pile cell is shown in Figure 2. A plan view of the extension is shown in Figure 3. As can be seen, no culvert is present because this plan will rely on the existing filling and emptying system of the 600-ft lock to fill and empty the extended 1,200-ft lock. Use of the existing filling and emptying system will require an extended fill time of the lock to maintain acceptable hawser forces. The lock floor is shown as a concrete slab on rock fill, which is described in the report as an aggregate filter. The floor slab is also to be structurally independent of the lock walls. The placement of the floor slab is not specified in the write-up on the drawings. For the purpose of this report, it will be

[1] A table of factors for converting non-SI units of measurement to SI units is presented on page viii.

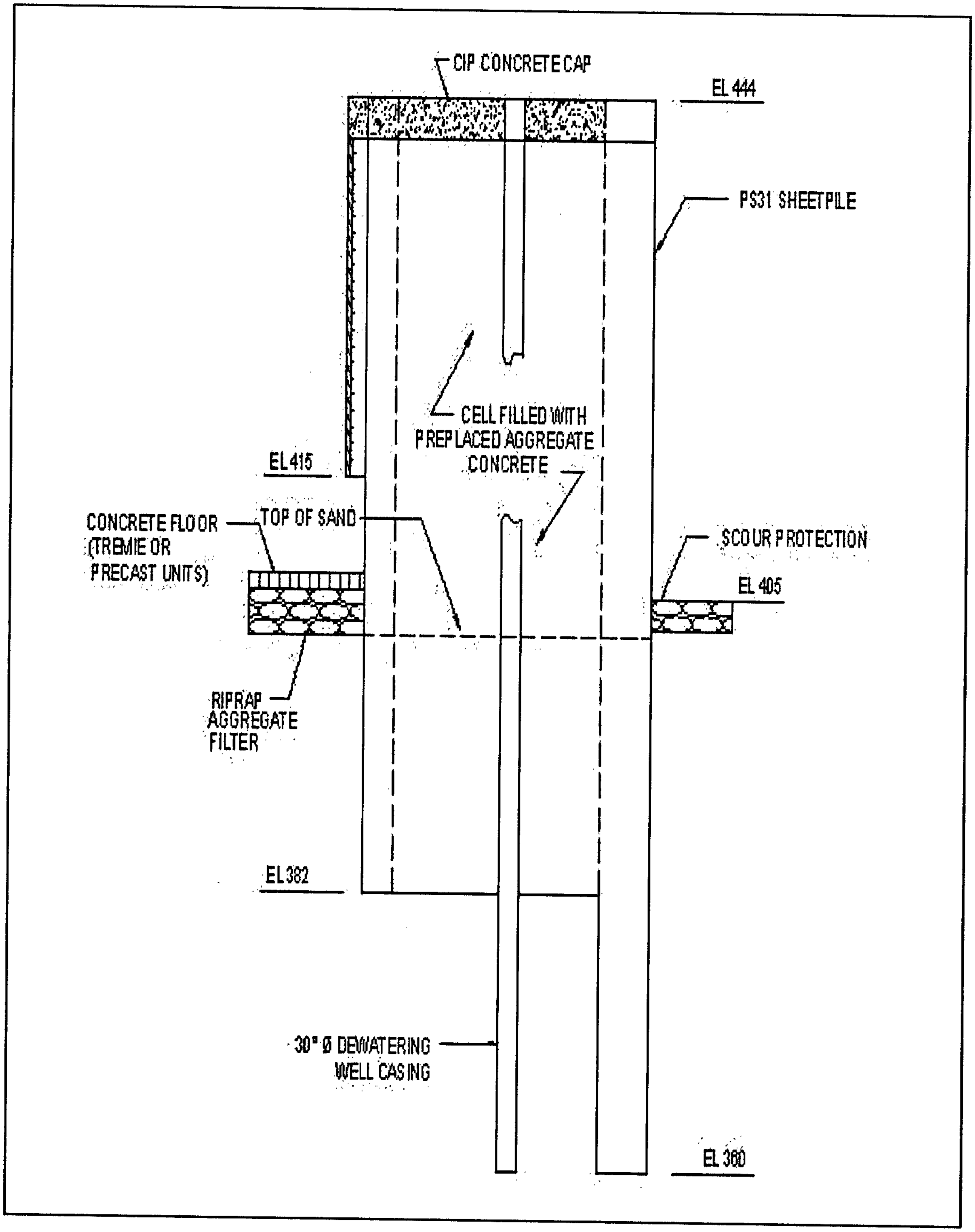

Figure 1. Plan 1, sheet-pile cellular wall used to extend 600-ft lock

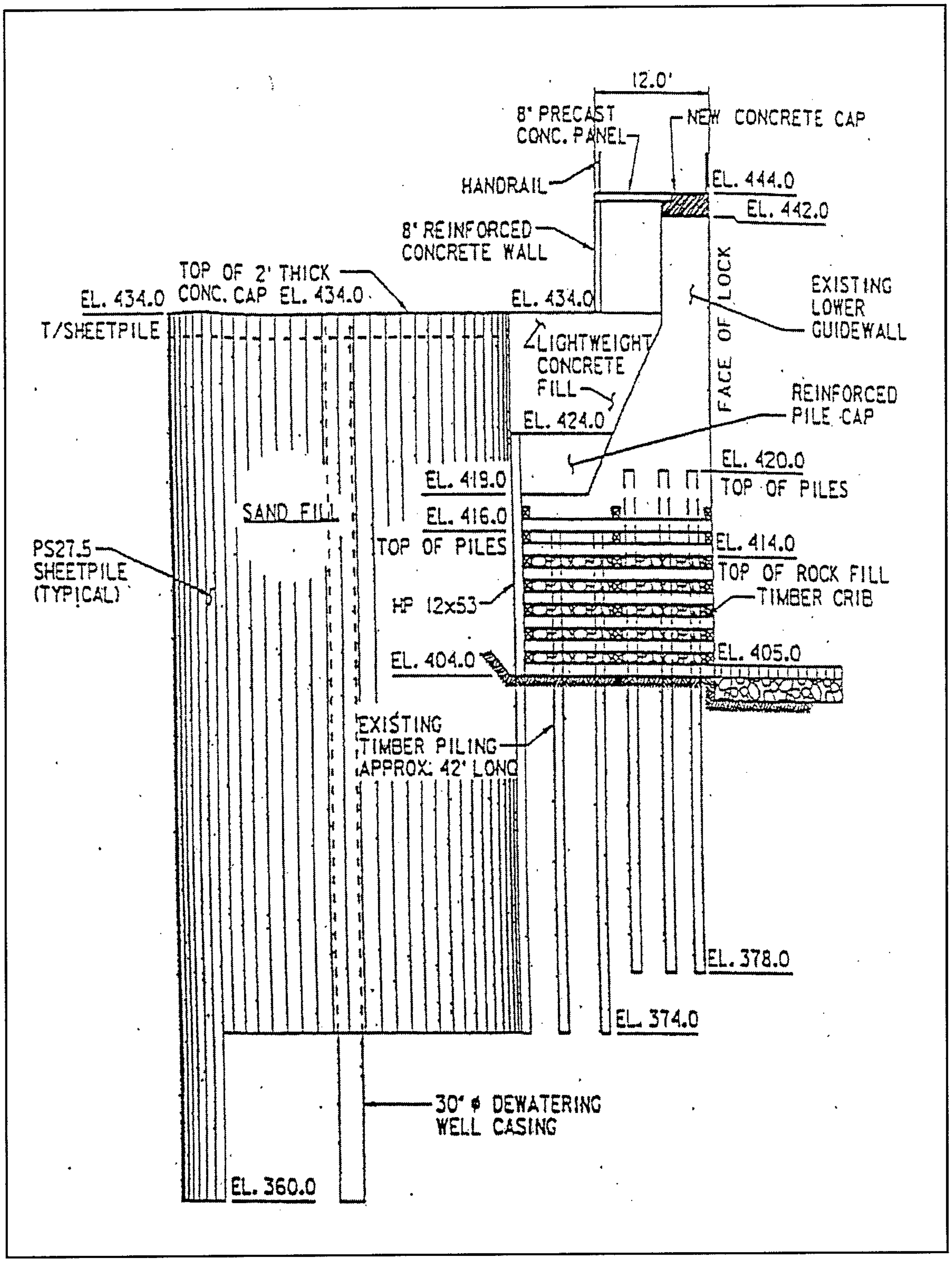

Figure 2. Plan 1, sheet-pile cells used to support existing guide wall

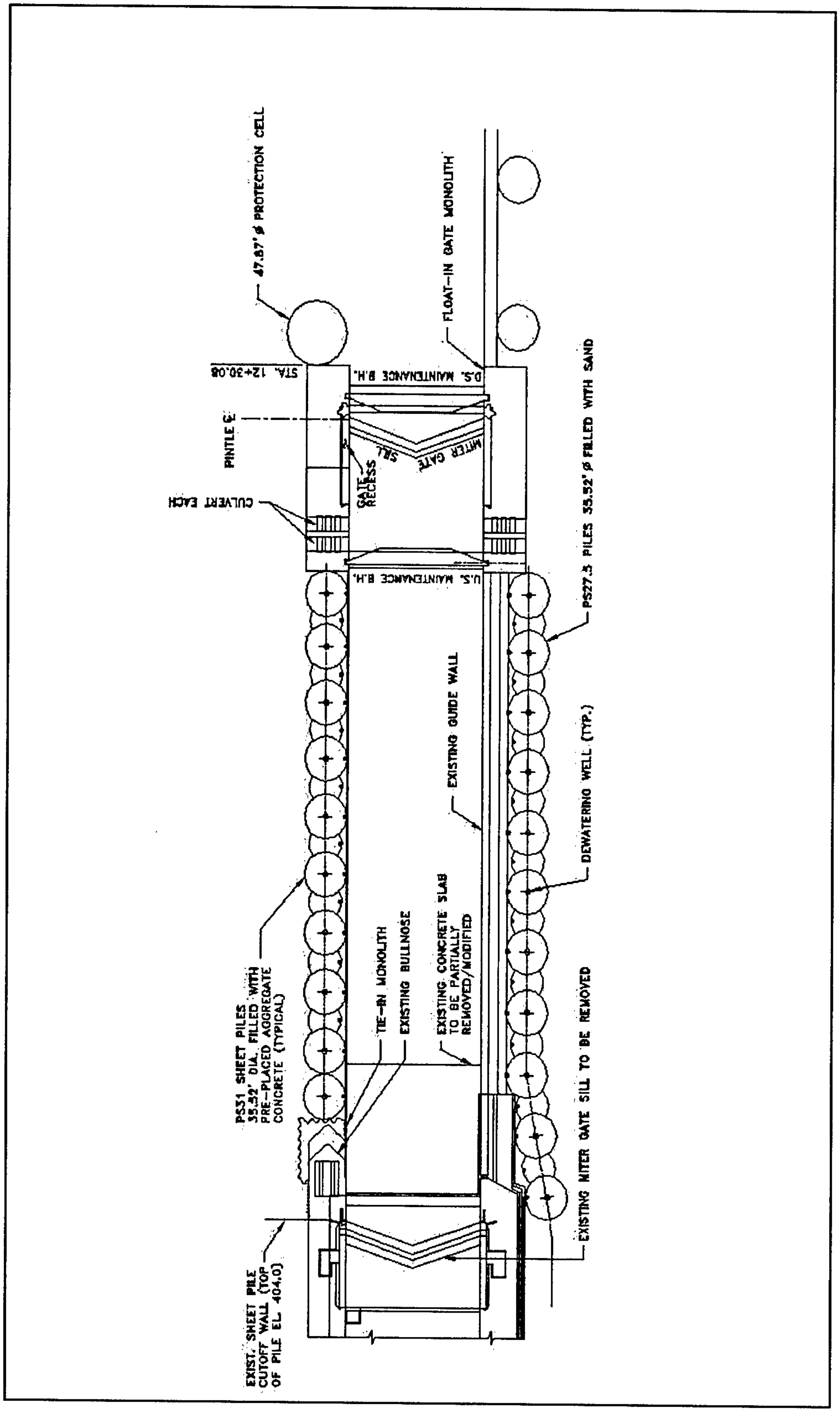

Figure 3. Plan 1, plan view of cellular wall extension of 600-ft lock

assumed that the floor slab could be constructed using tremie concrete or precast concrete slab units. If precast concrete units are used, some tremie concrete will still be required to fill in areas next to the sheet-pile cells. While the extended lock may be dewatered, it will be assumed that the floor slab will be placed in the wet to reduce the time required for construction.

Factors for consideration

Maintenance for Plan 1 should be minimal, although if precast units are used there is the possibility that displacement could occur. Because of the possibility of units displacing, a periodic inspection of the lock floor by a diver may be needed. Another area where there might be a possibility for concern when using the precast slab units is the scalloped areas near the sheet-pile cells which, because of thin sections, could also be subject to cracking and displacement. In general, however, maintenance for this plan should be very low. Stone protection placed in these areas at Temporary Locks 52 and 53 has performed satisfactorily.

With regard to implementation of Plan 1, there is likely not a simpler plan to implement. The slab may be constructed using tremie concrete or with precast concrete slab units. For tremie placement, the thickness of the placement may be an issue. Since the tremie pipe must be embedded in the concrete, there must be sufficient thickness to accomplish this. The concrete mixture itself becomes important, based on how level the slab is expected to be. Placing the precast units should not be difficult, although care will need to be exercised and reliable surveying techniques employed to ensure that the units are properly placed.

The plan as described in the report indicates that any construction that will interfere with the barge traffic will be done during the winter months when the UMR-IWS is closed to traffic due to ice conditions. Thermal stress analyses should be made if tremie concrete is to be used in the cold water. These analyses may be used to indicate if unfavorable stresses that may lead to cracking are present. In addition, the cold weather may affect set times of the tremie concrete. Use of the precast units would likely be a better solution with respect to a winter placement of the slab. Placement of the aggregate filter may also provide a challenge in an underwater environment, although riprap is often placed in underwater conditions. If placement of a filter cloth is required to limit the movement of fines, a special means of installing the filter cloth may need to be devised.

The risks to the operation of the lock are very small. The only possible problem appears to be piping that could occur if one or two well casings become inoperable, which could lead to an excessive load on the slab. Consideration for such a case should be made during the design and could be accommodated through establishing this as an extreme load case.

The construction time for the slab in this plan should be relatively short. Since it is independent of the wall, it can be constructed at any time during the construction process. The slab units may be a little more advantageous because the placement of the units may be done incrementally, with flexibility in the size

of the increments. Tremie placements could also be done incrementally, but some design issues (associated with the size of the placements) may need to be considered.

One limitation associated with this plan is that, since the floor is independent of the wall, the walls can gain no benefit from the floor slab as a tension tie or as a compression strut. Another drawback is that, should a problem arise with the deep well casings, dewatering of the lock will not be possible.

For a lock founded on rock, this plan becomes unnecessary since the rock will act as the floor of the lock. As noted in the description of Plan 1, this is intended for a lock extension. However, there is nothing associated with the lock floor that would preclude it from being used for new lock construction.

Plan 2

General

Plan 2 is shown in Figure 4 and was taken from the original UMR-IWS Study report for extension of a 600-ft lock to 1,200 ft. A plan view of this configuration is shown in Figure 5. The culvert is contained in the wall and therefore does not warrant any discussion in this report. Figure 6 shows an elevation of an alternative wall plan that was developed as part of the investigation, which has a culvert located outside the lock wall. The floor slab shown in both Figures 4 and 6 is a structural, cellular, precast, and prestressed section that will be floated into position and sunk into place through ballasting procedures.

The lock walls will have footings that will provide support for the floor system. The precast walls, including the footings, are founded on piles that provide the bearing support for the walls and any loads transferred from the floor slab system to the walls. HP 14 × 117 stub piles are used to tie the precast lock floor to the footing of the precast lock wall. The HP stub piles are embedded in the footing of the lock wall and extend into recesses in the precast lock floor. Once the floor system is properly positioned, the space around the stub pile is grouted, as well as any space between the floor system and the lock wall to ensure that the floor will act as a compression strut. The floor system and its connection to the lock wall will also need to be designed to carry tensile loads when the upper pool is in the chamber and there is a low tailwater. The stub piles will have to be designed to resist uplift loads during dewatering. Seals will need to be included at the joints of the floor system to prevent leakage during dewatering.

Once the floor system is in place, the area underneath the slab unit will need to be grouted to eliminate areas where flow beneath the floor system might occur. This grouted area should not be counted as providing structural support. Detailed analyses may show that piles driven in spud wells are necessary to provide

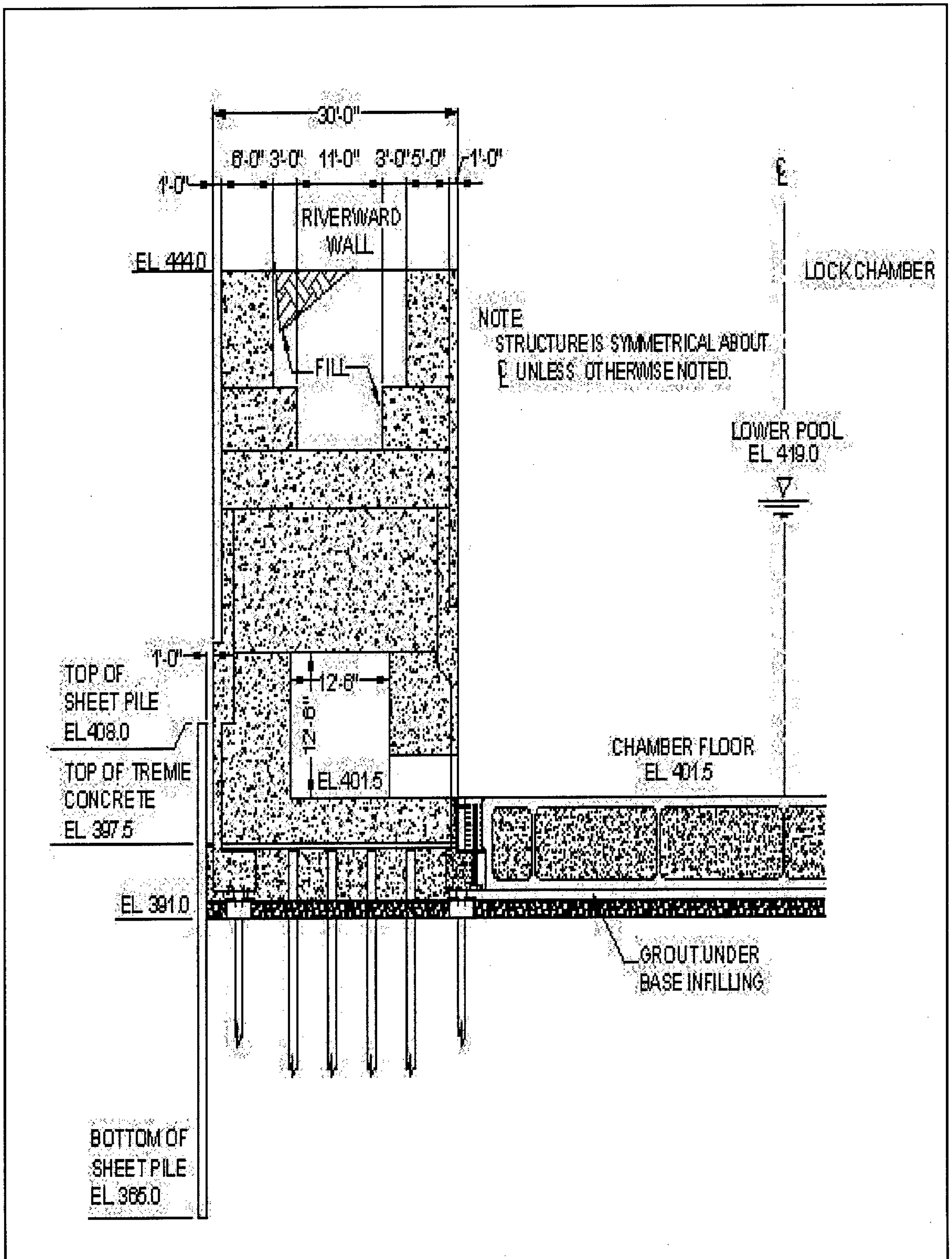

Figure 4. Plan 2, precast cellular structural lock floor

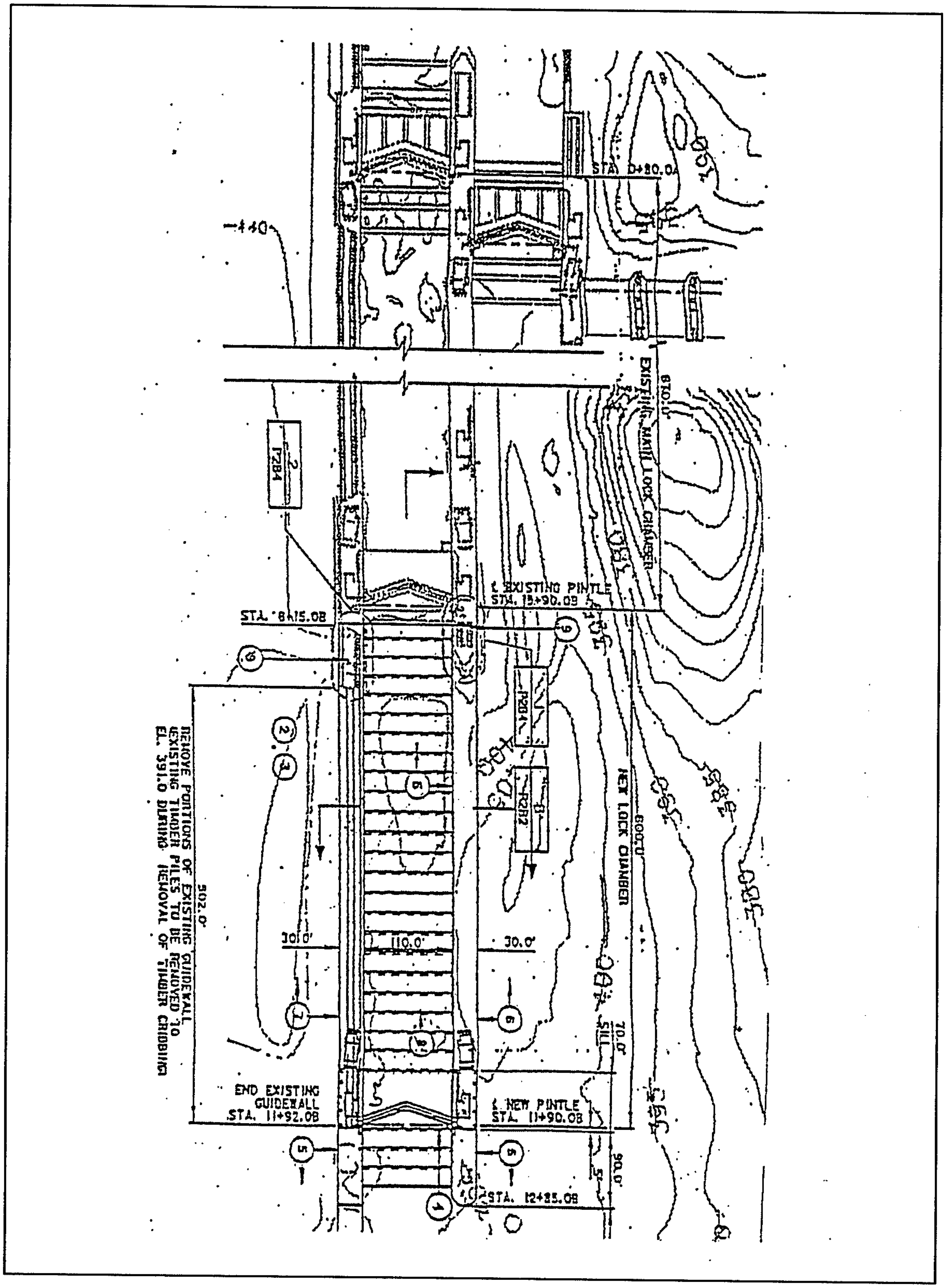

Figure 5. Plan 2, plan view of 600-ft lock extension, structural lock floor

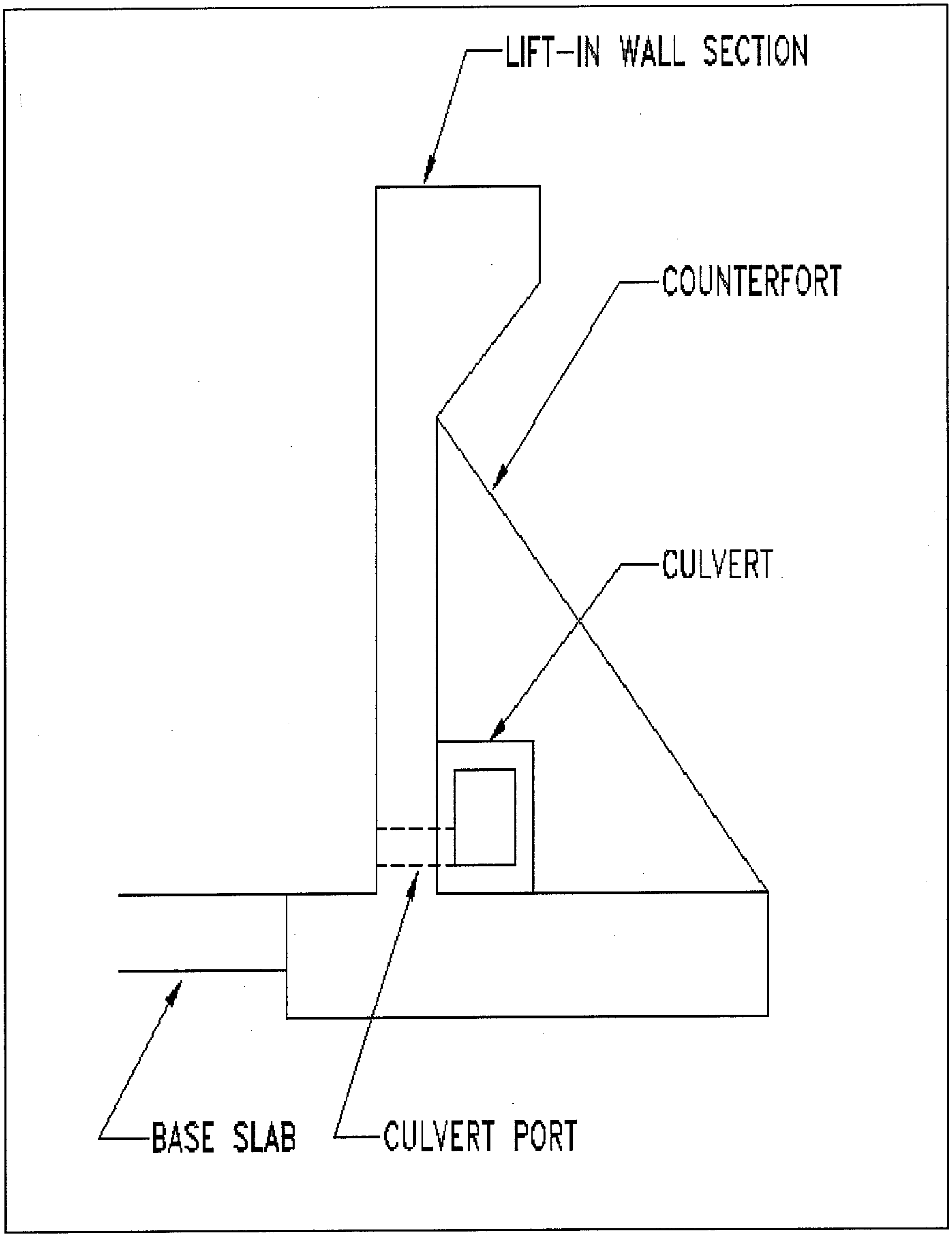

Figure 6. Plan 2 alternate, lift-in-place segmental wall with precast culvert

additional support for the floor system. Following the grouting under the floor, the chambers within the slab unit will need to be filled with tremie concrete. This slab unit will act as both a tension tie and compression strut between the lock walls, depending on the loading that is being applied. Since it is an integral structural member of the lock, the connection between the slab unit and the lock walls is critical.

Prior to placing the floor system, the elevation of the river bottom will need to be excavated to a given elevation using techniques to achieve this elevation, as described in Chapter 4. Also presented in Chapter 4 are discussions about how to maintain the excavated foundation. An 18-in. layer of scour-resistant leveling stone will then need to be placed on the foundation to help maintain the excavated foundation.

As noted, the plan shown in Figure 6 is an alternative plan. In this plan the wall unit is a lift-in-place unit with thin walls that are intermittently stiffened by counterforts. The counterforts would need to be constructed to accommodate the culverts, which pass through the counterforts. The filling and emptying ports go through the walls; thus, these ports would also need to be included during the fabrication of the wall units. The culvert sections would have ports in them to match those in the walls. Details on how the culverts and walls fit together will be discussed further in Chapter 4.

It should also be noted that the culvert shown in Figure 6 could be used with a variety of filling and emptying valve systems. It could easily be used between conventional valve monoliths. A special valve unit may also be designed for use within the culverts. The inlet and outlets that lead to the valves should be similar to those used on existing projects.

Factors for consideration

Since the slab is the same in Figures 4 and 6, although the culvert arrangement is different, discussion of the slab will be done independently. This discussion will be followed by a discussion about the culvert shown in Figure 6. The culvert shown in Figure 4 is part of the wall construction and therefore will not be considered here.

Slab unit. Maintenance for this particular slab should not be required and should provide a good surface during lock dewatering. In addition, there should be no risk to the operation of the lock for this plan.

The actual construction for this plan is significantly more involved than for the slab discussed in Plan 1. Installation will be slower due to the fact that the slab unit has to be ballasted into place, a connection made with the lock walls, grout placed beneath the unit, and chambers in the unit need to be filled with concrete. Experiences at the Olmsted Dam and Braddock Dam projects should provide valuable insight to floating and lifting precast units into place. It is also likely that a diver will be needed to check the connection to ensure it has been made properly, unless a method for remotely checking the connection can be

devised. Preparation of the foundation will not be as critical for this plan as it may be for Plan 1.

The cost of the slab in Plan 2 will certainly be significantly more than the slab in Plan 1. The prefabricated unit will require much more forming and is to be prestressed. In addition, there will be costs associated with making the connection, grouting beneath the slab units, and filling the chambers with tremie concrete. Direct comparison of the slab systems in Plans 1 and 2 heavily favors Plan 1. However, if the benefit of the slab system to the design of the lock wall in Plan 2 is incorporated into the cost comparison, the comparison between Plan 1 and other plans to Plan 2 will be more equitable and, possibly, Plan 2 will provide a less expensive solution.

The placement of the slab unit was discussed previously, and it was noted that the time to install the system would be longer than for Plan 1. If this system is being placed into a configuration (such as a lock extension) where traffic would be interrupted, it should be noted that the system is placed in sections and therefore could be placed in increments. This would allow traffic to pass through, in between the closures that would be necessary to install the slab units. To further ease the impact, the necessary closures could also be scheduled by doing much or all of the work during winter months when river traffic is light or the river is closed to traffic.

The slab unit shown in Figures 4 and 6 is limited to certain wall types. For instance, it could not be used in association with a sheet-pile cell wall system. The connection made between the wall and the slab unit will require strict tolerances; therefore, a detailed design will be critical to ensure that the connection can be made in the field. Another part of the connection that will require scrutiny in the design is ensuring that uniform bearing occurs at the connection.

The slab unit as shown in Figures 4 and 6 is obviously for a lock founded on piles, and the slab unit would not be necessary for a lock founded on rock. This system could be employed for slabs in the extension of an existing lock and the construction of a new lock.

Culvert. Maintenance of the culverts such as the one shown in Figure 6 should not entail any more than a conventional culvert. If concern exists that there is a greater risk for damage from debris, replaceable culverts can be designed and several extra units could be kept onsite for replacement since they are in sections. Such damage should not affect the operation of the lock unless damage occurs to culverts on both sides of the lock. This is not any more likely, though, than valves malfunctioning on both sides of a conventional lock.

Installation of the culverts can likely be accomplished without tremendous difficulty. The size of the crane needed to lift and position the culverts should be much smaller than what would be required for many of the other components of such innovative projects. To aid in positioning during installation, guides can be placed on the wall unit and on the culvert. Connections can also be designed at

the counterforts and the ports to make sure that openings in the wall are in alignment with those in the culvert.

The cost of these culverts may be more than a typical precast culvert due to the connections that need to be made with the wall unit. Ensuring proper alignment may require some special embedded steel fittings. While this aspect of the culvert will be costly, it can be seen that there should be significant savings in the walls that would be used with this culvert. The overall cost of the wall and the culvert would need to be compared to make an appropriate determination about the cost.

The hydraulic efficiency of this culvert system should not be affected by slight discontinuities at the location of the connection of separate sections, since the discontinuities are small and the head differentials at most locks on the Ohio and Mississippi Rivers are low-head locks (30 ft or less). However, a careful evaluation of these types of connections should be made during design to minimize erosion of the concrete due to the presence of slight offsets that might occur.

The time required to install the culvert sections should be relatively short. This required time may be shortened, however, by placing more than one culvert section at a time. The culverts do have the advantage that if the lock is an extension, placement of the culverts will not interfere with traffic. This would likely result in a benefit in the system where the extension of lock is being done.

The wall systems that can be used in conjunction with this culvert system are limited. Only precast wall systems providing for the culvert ports can be used for this culvert system. As with the slab unit, tolerances will be an important issue. Careful design of the connections should provide a means of overcoming any difficulties in this area.

The culvert shown in Figure 6 should be applicable for locks founded on piles or rock. This system would be best suited for construction of a new lock; however, it may be possible to use this arrangement in a supplemental system to the extension of an existing lock. If used as a supplemental system for a downstream extension of an existing lock, the culvert may need to be extended along the outer wall of the existing portion of the lock to provide an intake in the upper pool.

Plan 3

General

Plan 3 incorporates the use of precast concrete paving blocks in conjunction with a structural beam, as shown in Figures 7 and 8. This plan was taken from the report done for the UMR-IWS Study and was developed as a new lock system to be added to an existing upstream gate bay (built as part of the original construction). Several projects on the Upper Mississippi River have this

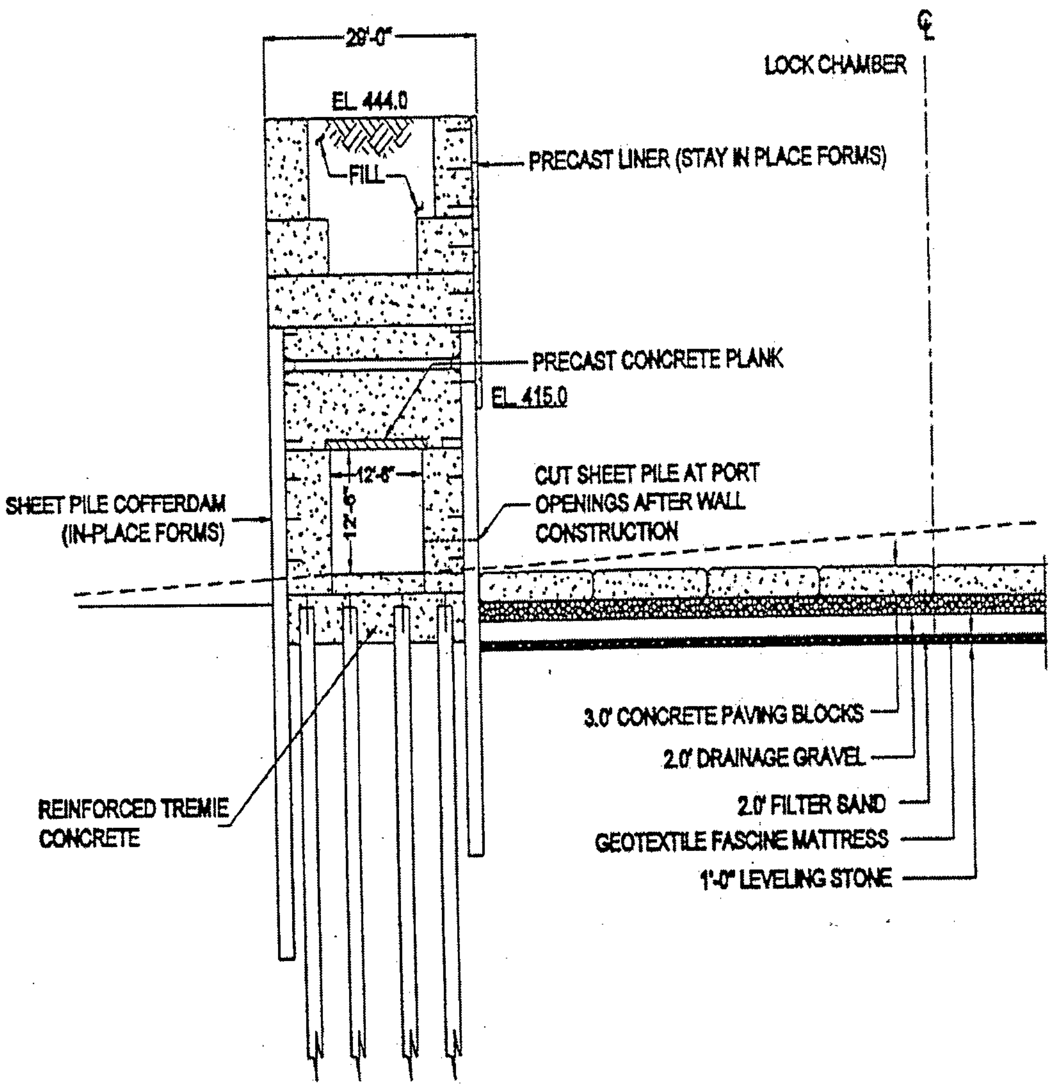

Figure 7. Plan 3, section through paving block lock floor, with bedding layers

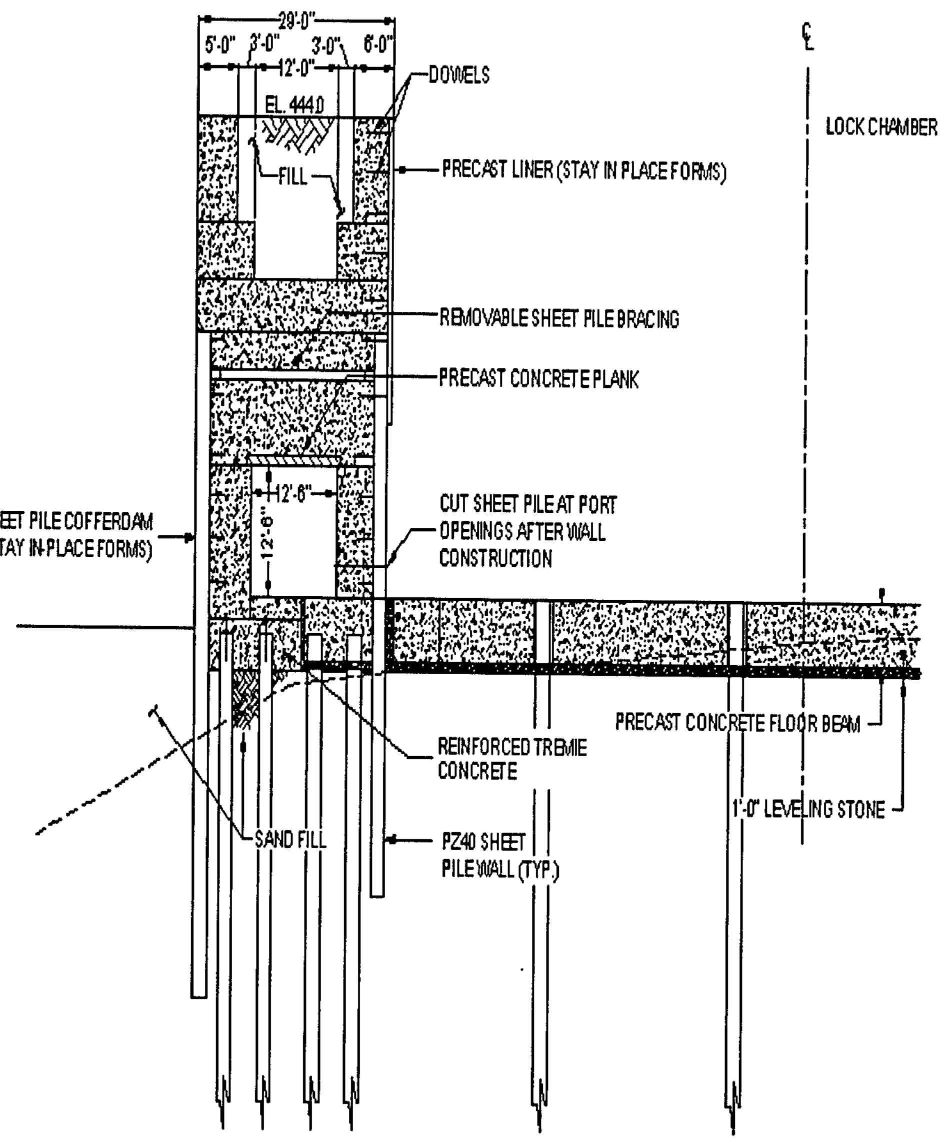

Figure 8. Plan 3, section showing interspersed precast concrete floor beams

configuration. A plan view of this arrangement is shown in Figure 9. The beam will be precast but does not span the entire width of the lock. A section between the beam and the wall exists which is intended to be used to splice reinforcing from the beam to the reinforcing extending from the wall. This is an important connection because the floor beam acts both as a tension tie and a compression strut between the two lock walls for improving the stability of the lock walls.

Alternatives to this connection plan may need to be considered for in-the-wet construction. This plan was to be undertaken in the wet; however, the connection was to be constructed in the dry. Review of this plan will approach this connection as if it were to be performed in the wet. The beams also contain spud wells that will be used as guides for driving piles. Once the piles are driven, the spud wells will need to be filled with grout to provide a connection between the floor beam and the piles. Methods for ensuring adequate bond between the piles and the beam will need to be developed, and possibilities for such connections are discussed in Chapter 4.

Precast paving blocks, approximately 14 by 15 by 3 ft with weep holes, will be used to pave the lock floor between the precast floor beams. A multistaged filter (consisting of leveling stone, a geotextile membrane, 2 ft of sand, and 2 ft of gravel) is to be used to prevent migration of fines in the sands underlying the paving blocks. Placement of the paving blocks and the base material (below the paving blocks) is to be made in the wet. Methods for accomplishing this are discussed in Chapter 4.

As with Plan 2, the culvert is part of the wall system and therefore will not be discussed here. However, the alternative plan discussed in Plan 2 could easily be adapted and used with Plan 3. The discussions about the culvert will not be included though, since these would be a repetition of the discussions for Plan 2.

Factors for consideration

As with previous plans, maintenance for Plan 3 should be low. Possible displacement of paving blocks is possible, particularly if weep holes become clogged. Therefore, keeping the weep holes clean will be an important maintenance issue. This item of maintenance becomes critical only during dewatering. In general, it does not appear that this plan poses any risks to the operation of the lock during normal operation.

The difficult aspect of this plan will be the installation of the floor beam and its connection to the wall. Since this beam is an integral structural component of the lock, it is critical that the beam is correctly placed and the connection is properly made. This connection may require the use of divers for inspection. While the placement of bedding material for the paving blocks and placement of the paving blocks themselves needs to examined closely to obtain the optimal method, the difficulty of performing these placements should not be great.

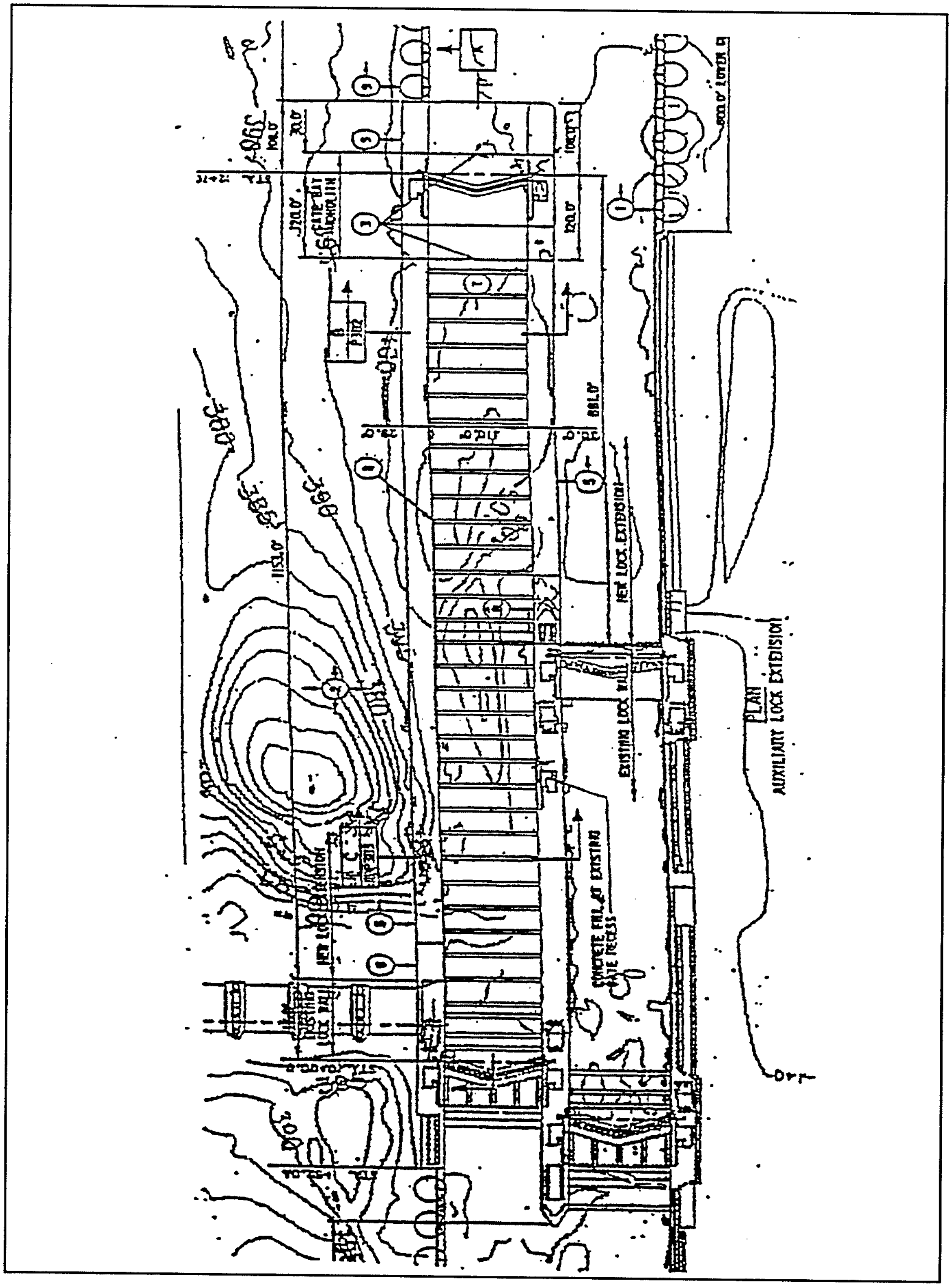

Figure 9. Plan 3, plan view of 1,200-ft lock riverward of existing lock

The cost of Plan 3 will be higher than Plan 1 but should be lower than Plan 2. This plan presents a viable compromise between Plans 1 and 2, in which the floor system is part of the structural system but less-expensive means of construction are used on a major portion of the floor. As with Plan 2, the cost of the floor system shown in Figures 7 and 8 will be higher than for the floor system shown in Figure 1 if a direct comparison is made. However, if the wall system is considered, the cost comparison may show that inclusion of structural members in the floor reduces the costs of the wall design such that it is the more economical plan.

As with implementation of Plan 3, the time to complete construction of the floor system for Plan 3 will likely be longer than for Plan 1 but less than for Plan 2. Placing the slab units to be used in Plan 2 is likely to take significantly longer than placing the beams and paving blocks of Plan 3. Similar to other plans, the floor system of Plan 3 can be constructed incrementally, to limit interference with barge traffic.

The limitations of this plan are similar to those of Plan 2 in that this floor system cannot be used with all lock wall systems, most notably sheet-pile cell walls. The tolerances needed for the connections will also be critical for this plan.

As with the other slab systems, a slab will not be required for a lock founded on rock. Plan 3 was presented as a pile-founded system. As discussed earlier, this plan is presented as construction of a new lock from an existing gate bay in the UMR-IWS Study report. However, it could be used for extension of an existing lock if interruptions to barge traffic could be eliminated or minimized.

Plan 4

General

This plan consists of a culvert system integrated with the floor slab system, and was also drawn from the UMR-IWS Study report. A cross section of the lock is shown in Figure 10, with a more detailed elevation of the slab/culvert system shown in Figure 11. This plan was evaluated in the UMR-IWS report as a new lock going through an existing dam, as shown in Figure 12.

The precast slab/culvert system is intended to be precast, prestressed concrete; is intended to be an integrated part of the lock's structural system; and is tied to the lock walls through reinforcing steel and cast-in-place concrete. Alternatives may be possible for achieving the connection between the float-in unit and the lock wall. The unit is intended to be floated into position and ballasted into place; piles are to be driven through knockouts cast in the section; tremie concrete is to be placed around the bearing piles; and finally, the areas around the pipes that are cradled within the unit are to be filled with tremie concrete. Prior to placing the units, leveling stone will be placed.

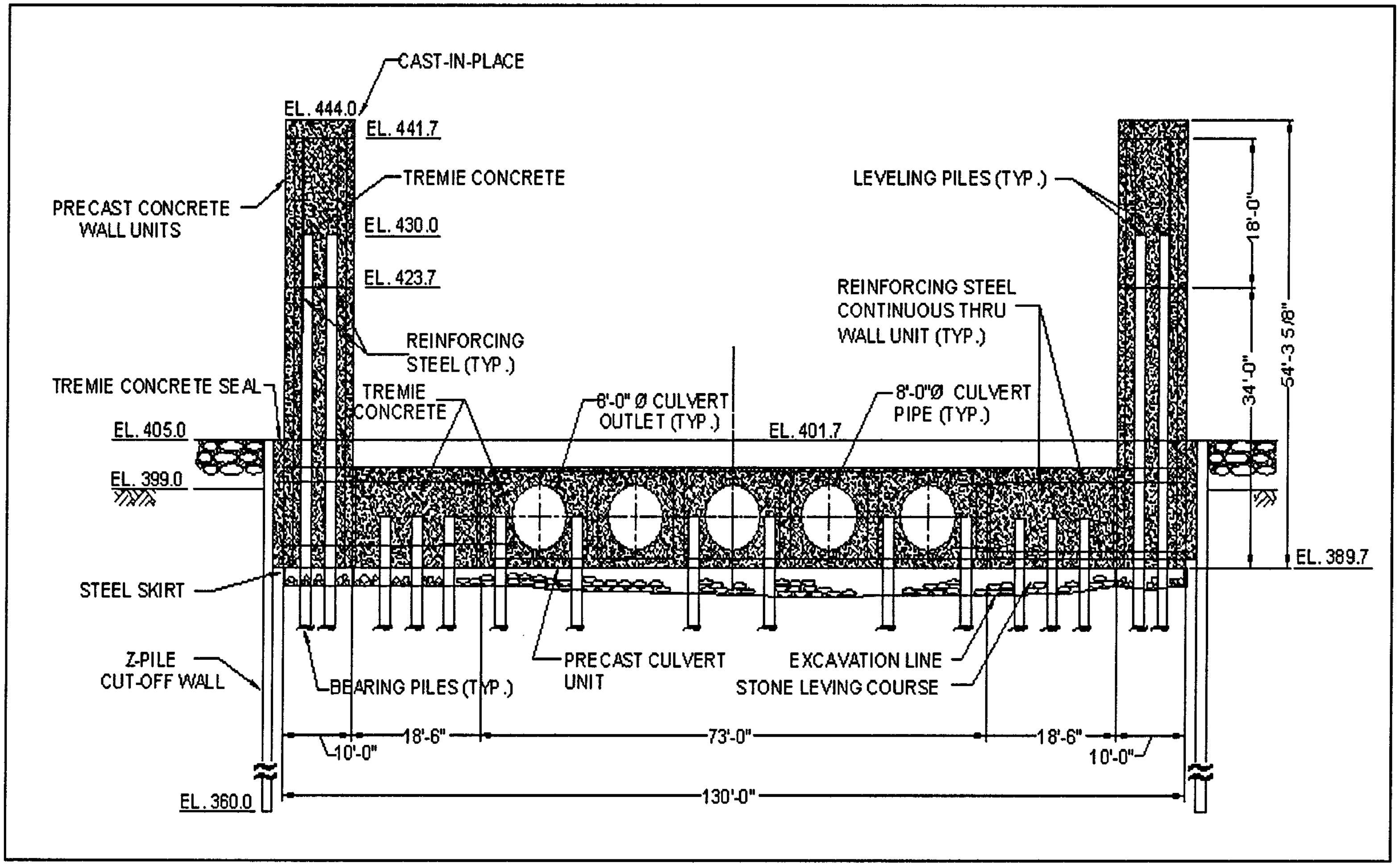

Figure 10. Plan 4, section through lock with precast slab/culvert system

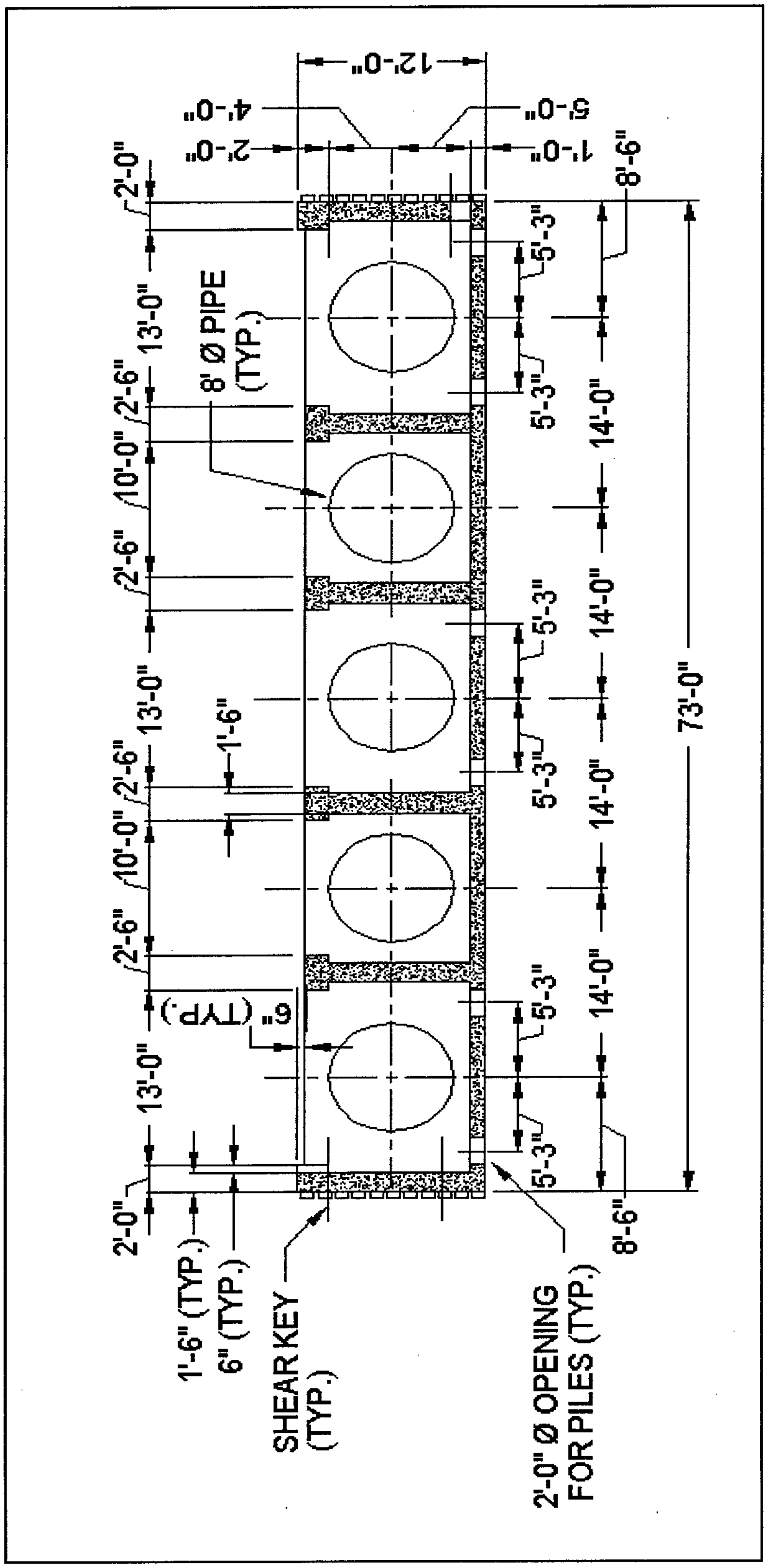

Figure 11. Plan 4, section through slab/culvert precast structural floor system

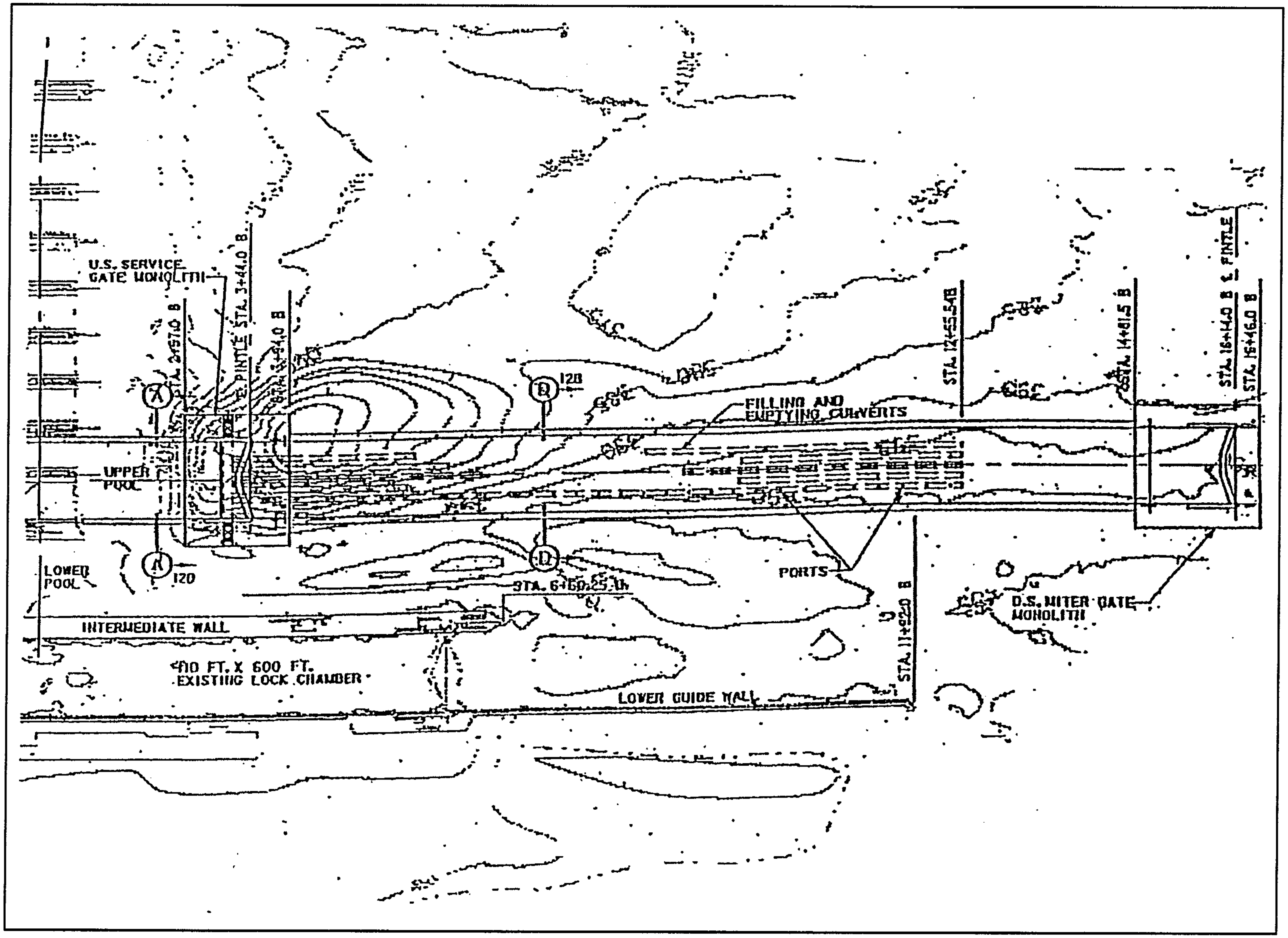

Figure 12. Plan 4, plan view of 1,200-ft lock through dam gate bay

Piles must be driven between the precast lock wall and the precast slab/culvert system. Once this is complete, a connection must be made between the slab/culvert system and the precast wall, and the area between the wall and the slab must be filled with concrete. The plan in the UMR-IWS Study was to lap reinforcing bars from the wall and the slab unit; however, other connection mechanisms may be possible and will be discussed in Chapter 4.

The filling and emptying plan associated with this system consists of butterfly valves at the upstream face of the upstream miter gate sill for filling of the chamber. When these valves are opened, they fill a mixing chamber contained in the sill. The culverts are fed from the mixing chamber, and the lock chamber is filled through vertical ports leading from the culvert pipes. For emptying the chamber, with the butterfly valves closed, two sluice gates would be opened to allow the water to flow out of the culverts, back into the mixing chamber, and through the manifold controlled by the sluice gates. This is possible because the upstream miter gate is downstream of the dam. A plan view and section of the proposed miter gate monolith containing the filling and emptying valves for this system are presented as Figures 13 and 14, respectively.

It should be noted that other filling and emptying schemes could be used for this culvert arrangement. Conventional tainter valves could be used that would feed mixing chambers at both the upstream and downstream ends of the lock, which would act as a feeder and distributor, respectively, for the culvert pipes. A variety of possibilities exist to accomplish filling and emptying of the chamber. Another item of note is that the current configuration shows the use of the vertical ports. It is generally accepted that vertical ports will not allow the energy of the water to dissipate properly during filling, creating excessive turbulence in the chamber. Therefore, the system would need to be revised so that the water comes out of the ports horizontally by using baffles or extensions of the ports that divert the water horizontally.

Factors for consideration

Due to the location of the filling and emptying ports directly on the floor of the chamber, the possibility increases of trash getting into the chamber, blocking or interfering with the filling and emptying ports. This could produce a maintenance problem. The likelihood of this occurring would be small, though, since the force of the water coming through the ports during filling should provide an adequate flushing system for almost any type of debris. The probability of problems due to debris may change based on any changes that may be made to the ports to dissipate the energy of the water during filling.

Ease/difficulty of placement. The risks to operation of this plan should be minimal, although there is concern over the effect of the vertical ports on small craft using the lock. Close investigation of this will be warranted during hydraulic model studies, should a decision be made to pursue vertical ports without baffles or redirection of the water flow. One advantage to the filling and emptying system as it is shown in the report of the UMR-IWS Study is that, if a butterfly valve

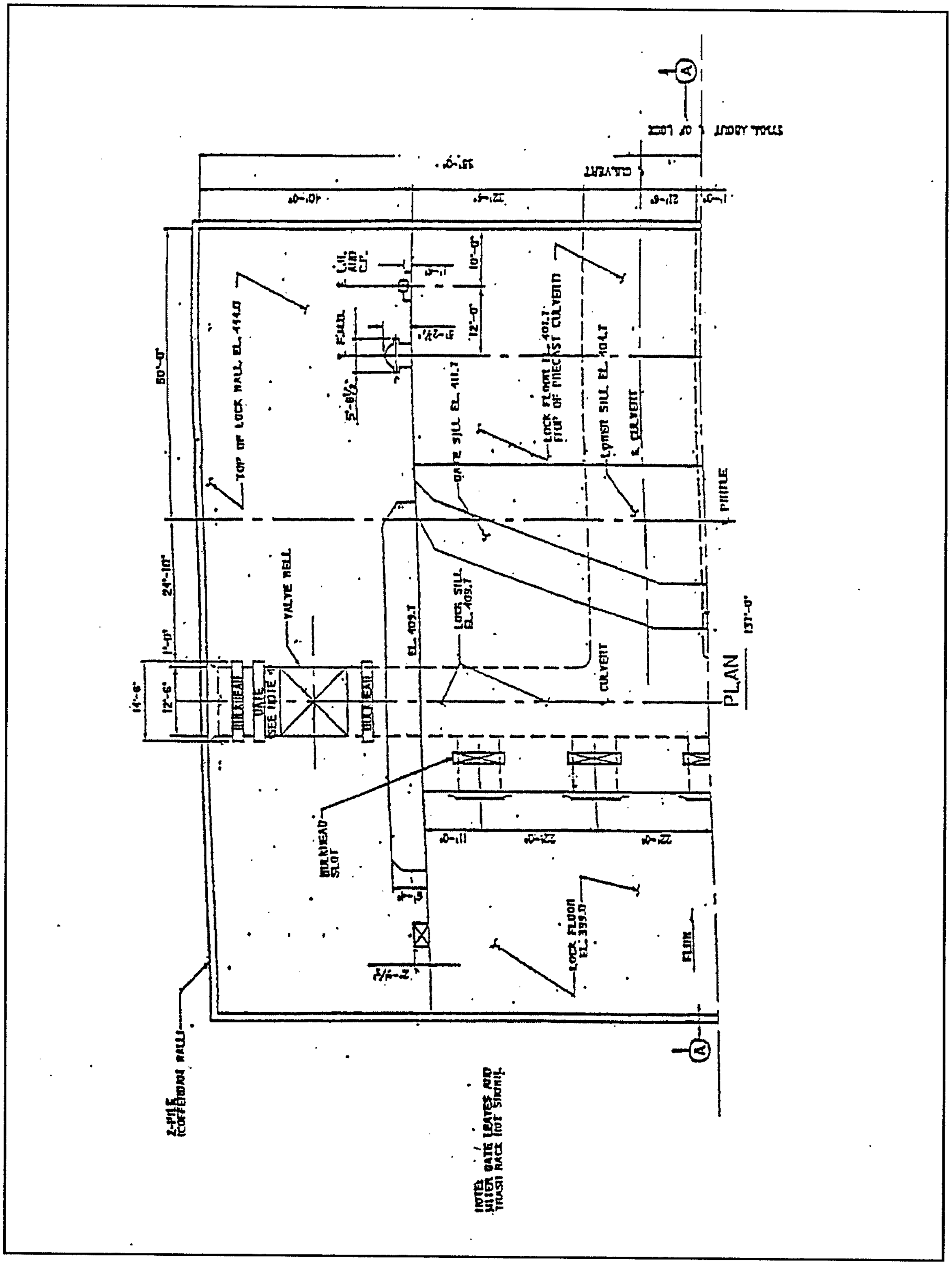

Figure 13. Plan 4, plan view of miter gate will with filling and emptying valves

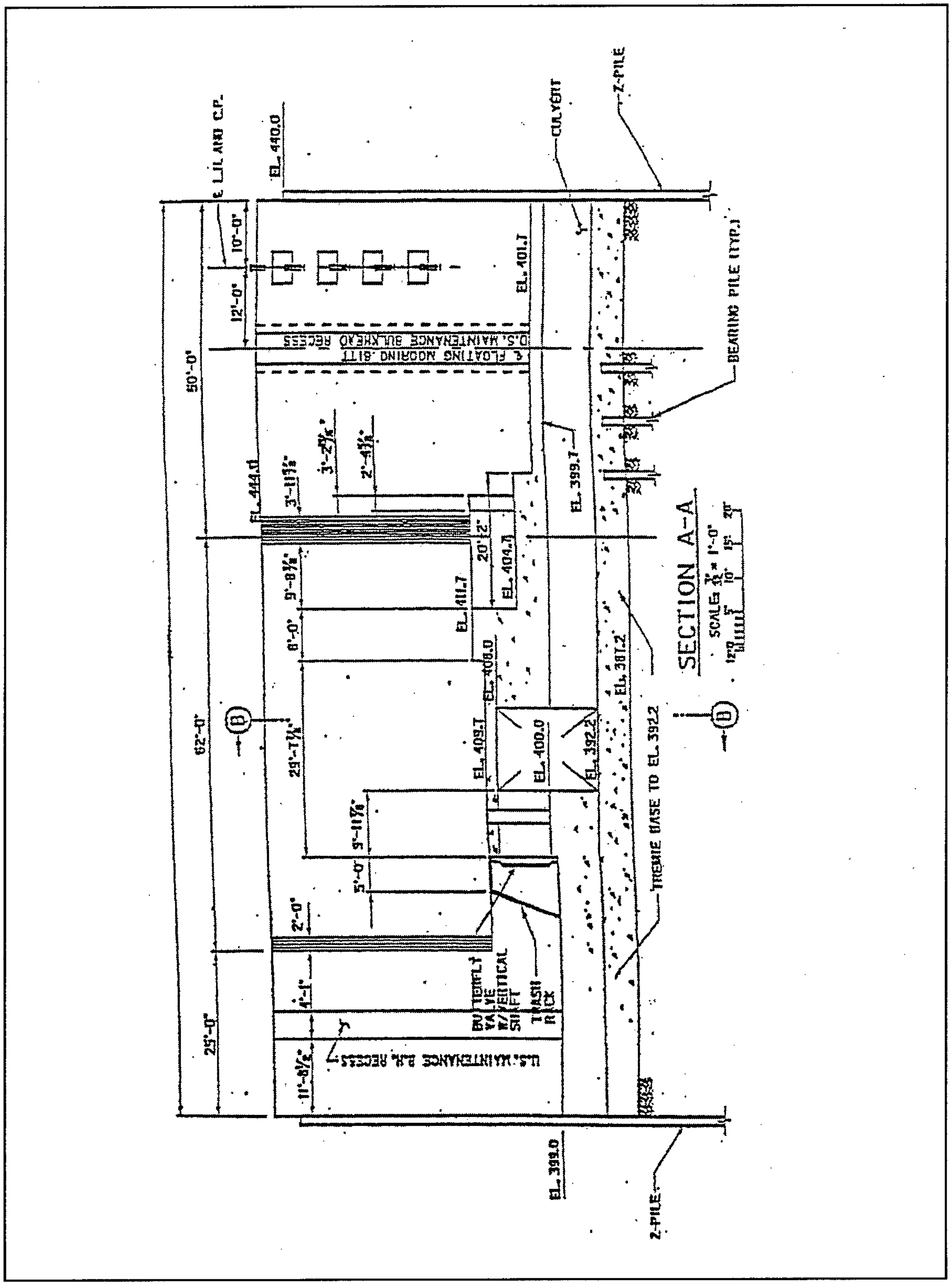

Figure 14. Plan 4, section through miter gate sill with filling and emptying valves

quits working, the impact on the filling and emptying system will be less than the impact seen on a conventional system when a tainter valve becomes inoperable. This should be a benefit for this filling and emptying system since there is some concern about placing valves in the sill. Although the system has redundancy with the multiple valves, these valves should be designed so that they can be removed as an entire component, and spare valves should be available at the site. The valves should be designed so that removal and replacement can be done quickly and, if possible, without the use of divers.

Plan 4 will likely be the most costly of the plans presented with respect to the slab itself. But as with Plans 2 and 3, its benefit to the wall system is also the greatest of the plans presented and would likely result in substantial savings for the lock wall construction. While it has been noted that the filling and emptying does not have to be through the sills, this culvert system is well suited to filling and emptying through the sills. This results in even further savings when compared with a system where the valves are contained in the lock walls.

The hydraulic efficiency of the proposed system is not completely clear. The right-angle turns that must be made from the culverts to and from the mixing chamber are not the most desirable from a hydraulic viewpoint. The vertical ports are likewise not the most advantageous arrangement for port systems, and in all likelihood the port system for this plan will need to be revised.

While the precast elements used in Plan 4 are somewhat larger and more sophisticated than the precast elements used in Plans 2 and 3, the time required to place these elements should not be significantly longer than the time to place the elements in Plans 2 and 3. As with the cost, reduced placement time might be better for the overall system when the walls are taken into consideration. The walls as shown in Plan 4 can likely be placed faster than the walls that are shown in Plans 2 and 3. The reason for the thin walls in Plan 4 is due partly to the thick slab and the additional stiffness it can provide to the walls. These walls would likely not be feasible for the floor systems described in Plans 2 and 3. The only drawback for this plan with regard to preparation and placement at the site is that some additional excavation may be required due to the thickness of the slab when compared with the other slab systems.

As noted above, this plan was developed for a concept that would be a totally new lock. This plan is certainly best suited for new construction, but consideration could be given to using it for a lock extension if a supplemental filling and emptying system is planned. Use in such a situation would likely require additional culverts on the exterior of the existing lock to be able to reach the upper pool for the filling cycle.

As with the other systems described previously, a slab is not needed for a lock founded on rock. For this plan, however, since the culverts are part of the slab, it may be reasonable to investigate the use of this precast element for the culvert system. Use of this plan for a lock founded on rock may also require a different wall design.

Plan 5

General

Plan 5 is another plan that was originated in the UMR-IWS Study. It is an in-chamber culvert system, as shown in plan view in Figure 15 and in section in Figure 16. The intent with this plan was to construct a new lock through an existing pair of dam gate bays. This plan incorporates the use of cellular sheet-pile walls, similar to those used in Plan 1.

The culvert system will consists of rectangular-shaped box culverts, as shown in Figure 16 that may be floated or lifted into place. Connections of the culvert sections will need to be established so that the connections can be made easily, a watertight seal can be maintained between sections, and the tolerances for lining up the culvert sections are small. Prior to placement of the culvert sections, piles will need to be driven or the soil stiffened to provide the proper support. Grouting beneath the culverts will be needed to provide a connection to the piles if it is the foundation system used and to provide uniform bearing if a soil-stiffened foundation system is used. As can be seen in Figure 15, the current configuration of the culverts shows a vertical port system. As discussed previously, this is not likely to be used because the energy as the water exits the ports cannot be adequately dissipated. Alternatives to this plan will be considered for the port system, such as baffles to redirect the flow. The filling and emptying system to be used in this plan is the same as the one described for Plan 4. The miter gate monolith containing the filling and emptying valves is shown in Figures 13 and 14.

The floor of the lock adjacent to the culverts will be composed of riprap. Beneath the riprap, the foundation will be prepared with a 5.5 ft bedding of filter material. The culverts do not extend the length of the lock, as can be seen in Figure 15. Concrete paving blocks are used downstream of the culverts. Weep holes will be included in the paving blocks to allow certain prescribed amounts of seepage, and will prevent excessive uplift on the paving blocks from occurring. The cellular walls contain deep wells to reduce or eliminate any seepage that may occur during dewatering.

Factors for consideration

The possible movement of riprap may be a maintenance concern for Plan 5. This is a concern for several reasons, the first being that if the riprap moves around, it may need to be regraded or additional riprap added due to settlement. Riprap moving around may also be a concern with regard to interfering with opening and closing of the miter gates. A large piece of stone could at a minimum cause closure of the lock due to interference with miter gate closing and could possibly cause damage to the miter gate. A solution to this problem may be to add a weak grout or a soil-cement mixture after the riprap is placed to hold it in place. Without this measure, periodic diver inspections would be warranted for Plan 5.

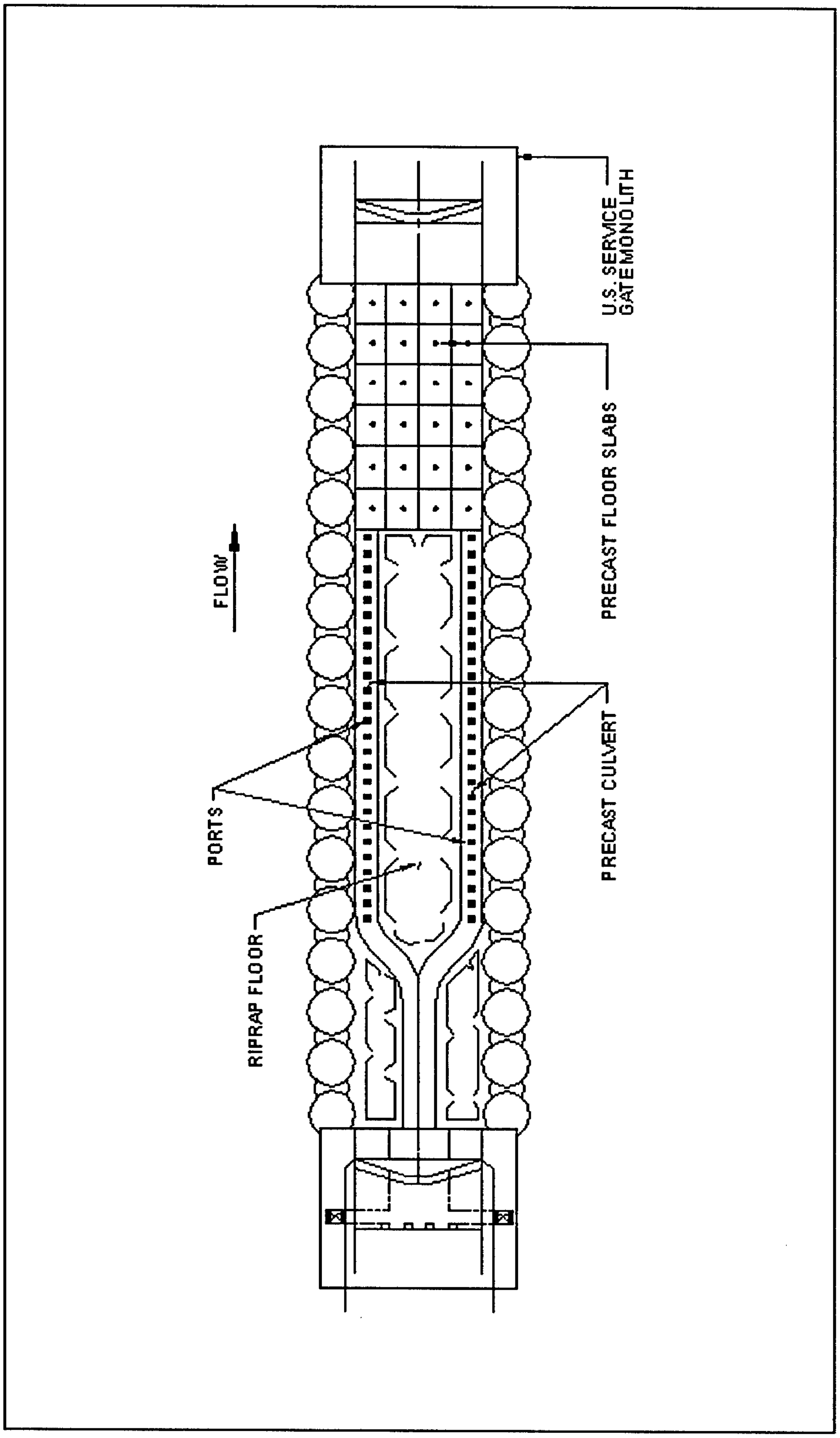

Figure 15. Plan 5, plan view of cellular locks walls with culverts in chamber

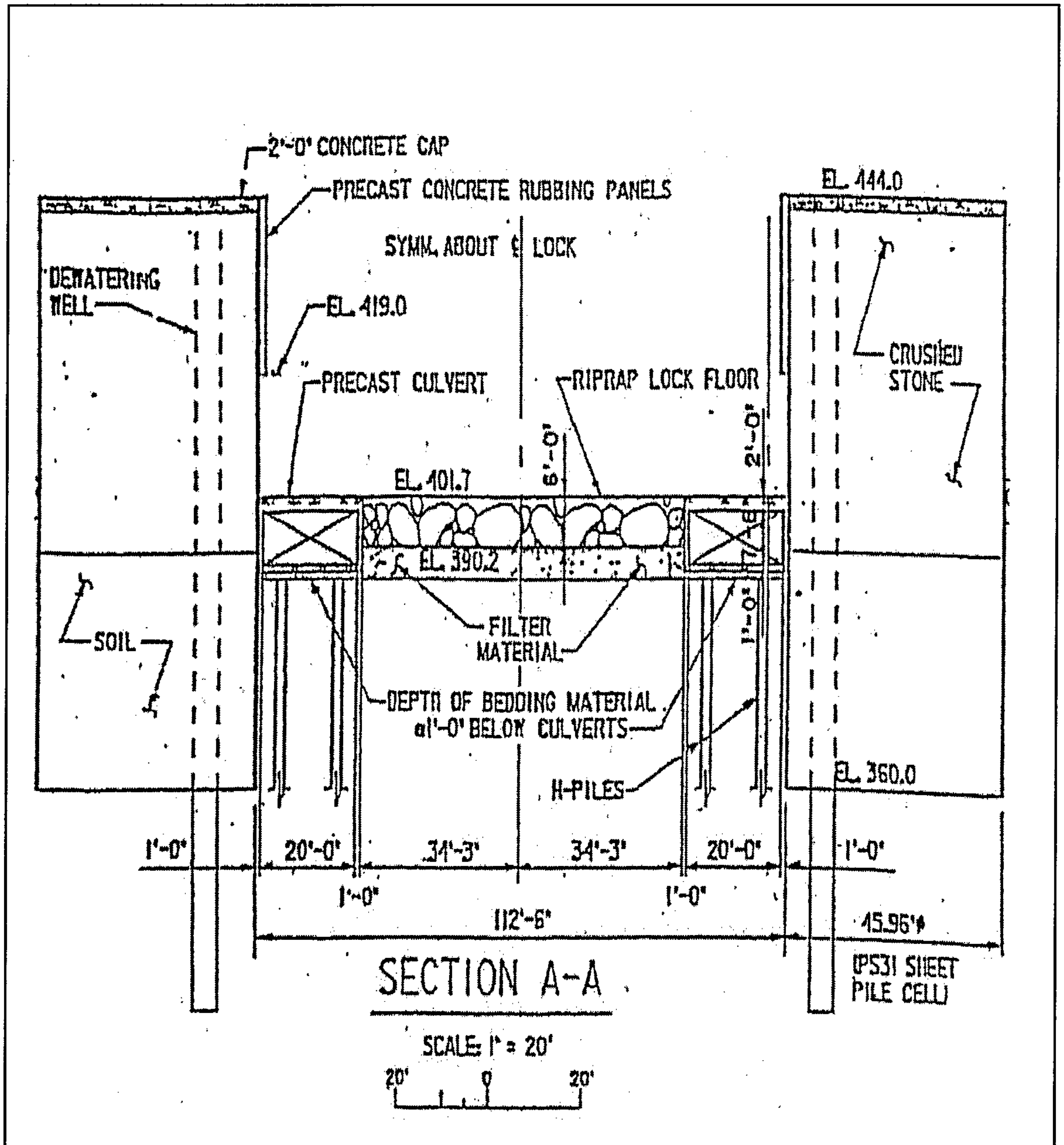

Figure 16. Plan 5, section of cellular lock walls with culverts in chamber

While plugged drain holes in paving blocks may be a maintenance problem, paving blocks are placed with open perimeter joints 2 to 3 in. in width. This should alleviate the consequences of plugged drain holes during dewatering. Operation of a vertical port system will require some monitoring of filling and emptying times to ensure that ports are not blocked with riprap.

In general, the implementation of this plan should not be extremely difficult. If connections of culverts are well designed for the underwater construction, then getting the culvert sections placed should be straightforward, particularly since the culverts are placed against the chamber walls. Depending on the design of the culvert sections, the interaction of the culvert sections with the piles may be an important aspect. Providing a mechanism to grout beneath the culvert sections may be an important issue and, again, the ease of accomplishing this depends on making proper provisions during design. Placement of riprap should not pose any problems either. However, care must be taken so that it is not dumped on top of the culverts. Placement of paving blocks will be a basic operation and should not be a great concern.

The risks of operation of this plan appear to be minimal. As noted above, there is concern about the movement of riprap and the fact that, if it does move around, it could possibly interfere with operation of the miter gate. Also, as in Plan 4, the filling and emptying system might in fact be advantageous since, for the filling cycles, there may be five or six butterfly valves and if one butterfly valve becomes inoperable, the impact to the filling cycle is less than if a conventional tainter valve stops operating. With vertical ports there is also the concern of the level of turbulence in the chamber that occurs and its effect on small craft.

The slab system being used is definitely the least expensive of all of the soil-founded plans examined. The possibility of adding a grout to alleviate concerns about riprap moving around would still leave this plan as the least expensive alternative for floor slabs. Culvert costs would be similar to culvert costs in other plans, although smaller sections may be easier to place. As in Plan 1, the floor system provides no structural advantages for the behavior of walls and therefore may result in a wall system that is more costly than floor systems that include structural action by the slab.

Regarding the hydraulic efficiency of Plan 5, comments are similar to earlier plans. This system has some right-angle turns that are not highly desirable. Joint tolerances at connections of precast culvert sections should be kept small to reduce discontinuities in the culvert system, which can lead to erosion. The vertical port systems will likely need to be revised to provide some sort of energy dissipation of the water as it exits the ports. Most of these items can be resolved in model studies that will be performed for individual projects.

The construction time associated with this plan should be less than for most of the other plans. Placement of the culvert sections will be similar in time to placement of other float-in or lift-in units discussed previously. The placement of the riprap should be a quicker operation than any of the previous floor plans described.

Use of Plan 5 at a rock-founded site would be appropriate for the culvert portion of the system, although the grouting beneath the culvert sections might require a different approach than might be developed for a soil-founded site. The riprap floor system could be used for the extension of an existing lock. However, the use of the culvert system would require extra culverts to be placed through the existing chamber or on the outside of the existing lock so that the inlet could be placed upstream.

Plan 6

General

Figure 17 shows a plan view of Plan 6, which was taken from the Ohio River Main Stem—Prototype Alternatives Study. This concept involves construction of a new 1,200-ft lock on the landward side of two existing locks. Intakes for the new lock are conventional-type intakes upstream of the upstream miter gate. However, once the intake sections become culvert sections, the culverts turn into the approach chamber and the filling is performed through valves in the miter gate sill. The chamber is filled through two sets of side ports in the culverts. The culvert makes a 90-deg turn just upstream of the downstream miter gate and exits the culvert through a lock monolith, which contains the emptying valves. After passing through the lock wall, the culverts make another 90-deg turn and transition to pipe sections. The pipe sections then follow the downstream guide wall for approximately one third of its length before making another turn, which causes the culvert system to cross the approach of all three of the lock chambers. Except in the areas of the ports, the culvert and pipe are all completely embedded in a rock trench and then covered with concrete to prevent flotation during dewatering.

While some benefit may be realized by placing the in-chamber system using in-the-wet construction, the largest benefit that can be realized using this plan is to develop an underwater construction process for the section of the pipe that crosses the downstream approaches of all three of the lock chambers. The discussion will focus on this aspect of the plan.

Factors for consideration

The plan proposed as Plan 6 should result in a system that requires little if any maintenance. Depending on the depth of cover of concrete over the pipe, periodic inspections may be performed as a precautionary measure.

The most difficult aspect of this plan is the excavation of a trench in the rock to place the culvert pipe. Excavating a trench underwater will be a challenge, and

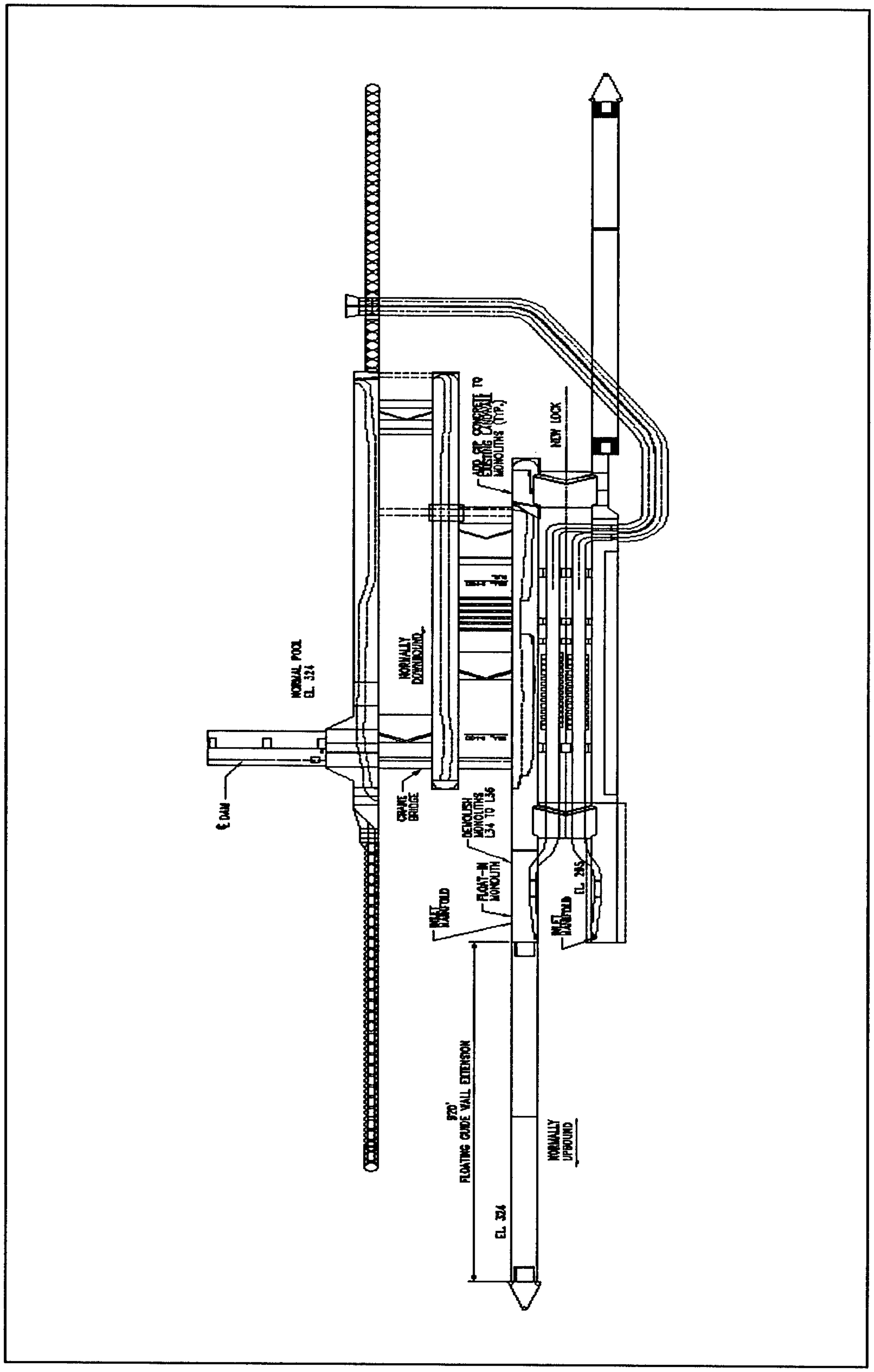

Figure 17. Plan 6, plan view of lock, conventional intakes, culverts in chamber

possible methods for doing this will be discussed in Chapter 4. Once the trench is complete, the culvert pipes must be placed. This should not be a major problem, as technology associated with the construction of immersed tubes can be adapted for this application. The other aspect of implementing this plan is the fact that the work must be done in the approach area of the existing (and operating) lock chambers. Careful examination of pipe sections will be need to be done to determine the optimum size, which will minimize the time required to place the sections. Smaller sections of pipe can be maneuvered more easily and quickly, but will require more closure periods; longer sections of pipe will require longer but fewer closure periods.

The risks to operation occur primarily during the construction process when the culverts have to be placed. Some closures may also be needed during the trenching of the rock and the subsequent placement of concrete around the culvert pipe. Another risk to operation would be that if for some reason the pipe becomes damaged, this repair could affect operation of all three locks. This possibility can be eliminated or reduced during the design process.

It is imperative that outlet culverts crossing the approach to an operating lock be placed using underwater methods. If the construction must be performed in the dry, it is likely that there will be a long closure period, which may be entirely unacceptable to the barge industry.

The hydraulic efficiency of the proposed culvert system should be good since this plan is similar to that used at the new R. C. Byrd Lock in the Corps' Huntington District. Concerns over losses and differentials of the two separate culverts, as well as over emptying of the lock, will need to be addressed through model studies.

The excavation process for the trench could be a lengthy process. Several alternatives for accomplishing this excavation may be available, and these are discussed in Chapter 4. Placement of the culvert pipes should be possible fairly quickly (i.e., limiting closures to less than 1 day) and, since they are placed in sections, can be scheduled in a manner that will minimize impacts on barge traffic.

Plan 6 is somewhat of a site-specific application since, if the new lock were built on the riverward side of the existing locks, the emptying culvert would likely not cross the approach of the existing locks. Therefore, this plan has limited applications but does provide valuable insight for those locations where it may be used.

Plan 7

General

Plan 7, which was also taken from the Prototype Alternatives Study performed for the Ohio River Main Stem, looks at the extension of the auxiliary lock. Figure 18 shows a plan view of the site, including the 600-ft extension to the auxiliary lock. This plan implements an auxiliary filling and emptying system for the extension. A culvert inlet is located near the upstream edge of the upstream guide wall. The culvert runs along the existing lock and, several monoliths into the lock extension, it turns and enters the lock where it becomes an in-chamber system. The culvert is a 16.5-ft-diam pipe until it transitions to a 16- by 14-ft culvert in the chamber. The culvert, in the center of the chamber, feeds laterals that extend perpendicularly to the culvert. Both the culvert and the laterals will be embedded in the rock and will require excavation of the rock. The culvert then splits prior to reaching the lower miter gate sill and exits through the lower miter gate sill where butterfly valves are used to control emptying and energy dissipaters located just downstream of the lower sill. The butterfly valves are to be constructed such that they may be removed for servicing without dewatering.

Factors for consideration

Any issues of maintenance associated with this plan will primarily be with regard to the valves located in the lower miter gate sill. It is preferable to have valves that can be removed in the dry or worked on in-place in the dry. Careful design of the valves would allow them to be removed as a unit and replaced without dewatering. This will substantially reduce any concerns about using this type of valves. Culverts and transitions must be adequately designed to resist seismic forces so they will not affect operation of the extended culvert system.

As in Plan 6, the most difficult aspect of implementing Plan 7 will be the process of excavating the rock underwater. Otherwise, placement of the culvert should not be particularly difficult except, possibly, where it joins the lower miter gate sill. Some special provisions may need to be made to ensure that the connection between the lower miter gate sill is easily made underwater.

The risks to operation of the lock are minimal, especially since there already exists a 1,200-ft lock. The fact that the downstream valves will be underwater could pose a problem if the main lock has to be shut down at the same time the auxiliary lock is shut down. This should be a rare occurrence and should not last for extensive periods, particularly if extra units are kept at the site to replace the valves contained in the sill.

The cost savings associated with performing in-the-wet construction of the extension of the 600-ft lock, as opposed to doing this work in the dry, will not be as significant as what can be expected for Plan 8. However, the maximum cost

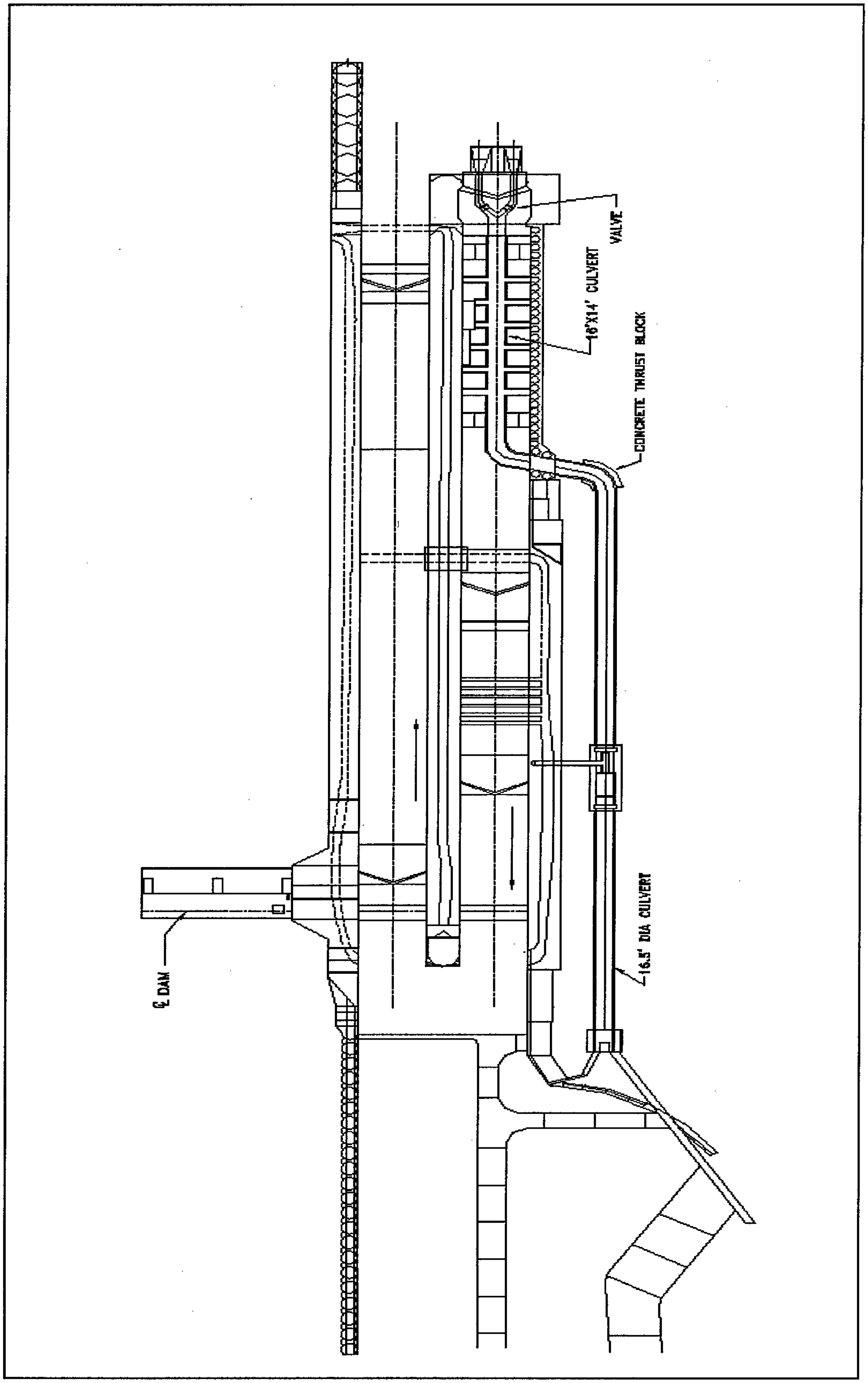

Figure 18. Plan 7, plan view of lock extension filled by external culvert

savings will depend on the method required to place the intake culvert, and this varies depending on site conditions.

As in Plan 6, the filling and emptying system will need to be evaluated through a model study since the extension of the lock will use an auxiliary filling and emptying system. Since the system for the extended portion of the lock is slightly different from that used in the original 600-ft lock, the system will need close study to determine the optimal filling and emptying times. The length of the culvert extending to the upper pool as well as the turn that this supply culvert must take to enter the extended portion of the lock will also affect the behavior of the system.

Constructing the lock extension in the dry, compared to in the wet, would likely take longer, but this may depend somewhat on the availability of the mechanisms for trenching rock underwater. This will be discussed in Chapter 4. The remaining aspects should be accomplished in a shorter period of time for in-the-wet construction than for in-the-dry construction.

Many aspects of Plan 7 are site specific, as was the case for Plan 6. Evaluation of this plan does provide an opportunity to review aspects of design and construction for extensions of rock-founded structures in more detail than previous plans discussed. This plan definitely applies only to lock extensions, but some aspects of the plan could be used for a lock founded on piles.

3 Design Parameters

Historically, navigation locks and dams have been constructed in the dry within cofferdams. Engineer Manuals and Engineer Technical Letters that apply to the design of navigation structures specify design criteria and design parameters, but do not specifically address methods of construction. Design criteria such as required loading conditions, required stability results, and performance requirements will remain unaffected regardless of methods of construction. Design parameters that may be affected by differing methods of construction, special construction materials, and lack of visual inspection will be evaluated for use in the design of the innovative lock plans described in Chapter 2. This study will be limited to lock floor paving, structural floor systems, and culverts that are constructed or installed in the wet.

Concrete Lock Floors

Concrete lock floor systems are used for a variety of purposes that depend on the foundation upon which they are placed and the loads to which they are subjected. Generally, lock floors on rock are not paved, unless the rock is extremely weak, and there is an identifiable source of erosion. On overburden, paving is used to prevent erosion of the lock floor and, with bedding materials, to control seepage forces that could remove fine sands and silts from the lock floor foundation. With the lock dewatered, underseepage forces have the potential to displace lock floor paving. Adequate pressure relief for these forces must be provided. Seepage cutoff walls constructed below lock walls extend the seepage `path and help control these forces. Filter or bedding material placed under lock floor paving limits seepage forces that could cause removal of fine sands and silts from the foundation. The weight of concrete paving resists seepage forces that reach the base of the paving. Weep holes provided in paving relieves seepage pressures by allowing seepage that has been reduced to a minimum to pass unrestricted. Concrete lock floor paving provided for these purposes will be constructed independent of the lock walls. There will be no connection between paving and the lock walls. Properly sized riprap over bedding material may also be used to protect lock floors against erosion and piping of the foundation materials.

Seepage analyses are important to the design of lock floor paving, filter-bedding, seepage cutoff walls, and foundation pressure relief wells, where required. Lock walls founded on vertical piles will often benefit from lateral

reactions provided by a properly designed and constructed lock floor system. In these cases, the lock floor system will contain members that are designed to resist tension or compression forces imposed by the lateral loads on the lock walls. By resisting these forces, the concrete lock floor system contributes to the stability of the lock walls. Dewatered locks will place rigid lock floor systems in compression. The normal loading conditions of the lock filled and the lower pool in the river will place floor systems in tension when the lock floor is tied to the lock walls by reinforcing steel or structural connections. Discussions below address parameters used for the design of lock floor paving and lock culverts, and the influence that construction in the wet may have on these parameters.

Concrete lock floor slabs on rock

General. Lock floors on rock are usually not paved unless the rock is considered erodible and forces exist that would tend to erode the rock. Concrete gravity lock walls will generally be founded on highly competent rock where practical. Lock floors will be excavated to provide the required depth of submergence for tows, or to reach a stratum of rock suitable for the support of longitudinal culverts, crossover culverts, laterals, or gate sills. Generally, water forces from ports or laterals, or from propeller wash, are not capable of eroding even weak rock. Any overburden within the lock chamber during construction will generally be removed to top of rock in lieu of paving. Water discharge over emergency lift gates that are used to pass ice and debris has been found to erode weak rock and thinly bedded shale. Such areas between emergency lift gates and miter gate sills will normally be paved with cast-in-place concrete. Paving will normally have a minimum thickness of 18 in. and be provided with contraction joints on 15- to 20-ft centers. As these slabs are subject to uplift, weep holes drilled 10 to 15 ft into rock will be provided for pressure relief. Such weep holes are generally assumed to be 50-percent effective in relieving potential hydraulic heads.

Anchors grouted in rock are provided to resist the excess uplift on the paving. Anchors are sized in accordance with the strength methods of the American Concrete Institute (ACI) 318, "Building code requirements for structural concrete" (1999), and in accordance with the requirements of Engineer Manual (EM) 1110-2-2104, "Strength requirements for reinforced-concrete hydraulic structures" (Headquarters-Department of the Army, HQDA). Embedment in rock is commonly based on the ultimate bond stress of grout on rock in accordance with the Post-Tensioning Institute's "Recommendations for prestressed rock and soil anchors" (1996). Assumed values for bond strength should be verified by load tests during construction. A hydraulic factor, H_f, of 1.65 will be used, as the anchors are in direct tension. In addition to this service factor, a load factor of 1.7 will be applied to the live loads and a load factor of 1.4 will be applied to the dead loads. Where deep scour holes have developed, removal of debris and placement of tremie concrete may be found to be an acceptable method of repair.

Factors affecting design parameters. Work accomplished underwater should be designed to provide the same level of protection against scour as work

performed in the dry. While this work can be accomplished in the wet, it will be advantageous to increase the minimum paving thickness to 24 in. to ensure adequate coverage of erodible rock. Reinforcing in the amount of 0.2 percent should be placed in the upper third of the concrete paving. This amount of reinforcing will be adequate to control cracking without the need for control joints. A self-leveling concrete mix should be used, and screeding should be required if acceptable dimensional tolerances cannot be met by the tremie process alone. Deep scour holes underwater are conducive to repair using tremie concrete. The process of cleaning the area to be filled is critical to a successful repair. A diver should inspect the underwater removal of debris. Screeding of the concrete surface is a construction technique that may be used if necessary to meet dimensional tolerances.

Concrete lock floor slabs on overburden

Concrete floor slabs may be placed on overburden when lock walls are founded on bearing piles or where cellular sheet-pile structures founded in overburden are used as lock walls. Lock floor paving may be cast in place, or precast paving blocks may be lifted and set into place. Soft alluvial deposits are removed to competent sands and gravels prior to construction of lock walls. Seepage cutoff walls are commonly driven beneath lock walls founded on bearing piles. The outboard sheets of cellular sheet-pile walls are normally driven to a sufficient depth to limit seepage forces to an acceptable minimum.

Seepage quantities and forces may be calculated once the coefficients of permeability of the foundation materials are determined. Filter beds placed under floor slabs are normally designed to prevent migration of fines, i.e., piping of foundation materials. This is accomplished by designing for appropriate factors of safety against the critical seepage escape gradients. EM 1110-2-1901, "Seepage analysis and control for dams," recommends the following factors of safety: FS $\geq$ 3.00 for usual loading conditions; FS $\geq$ 2.00 for unusual loading conditions, and FS $\geq$ 1.50 for extreme loading conditions.

Criteria for the gradation of the filter or bedding for the concrete paving is also given in EM 1110-2-1901. Basically, the filter must contain a 15-percent size sufficient to retain the 85-percent size of the foundation particles. Additionally, the filter must contain a 15-percent size sufficient to drain the 15-percent size particles of the foundation. Weep holes are provided in the lock floor paving to limit uplift on the paving due to head losses at the weep holes. Excessive water pressure under the lock floor paving has the potential to lift the paving. To ensure that this does not happen, flotation factors of safety from ETL 1110-2-307, "Flotation stability criteria for concrete hydraulic structures," are applied to the thickness design of the concrete paving. Factors of safety required are 1.50 for normal loading conditions, 1.30 for unusual loading conditions, and 1.10 for extreme maintenance loading conditions. Where analyses indicate there will be large quantities of underseepage with the lock dewatered, relief wells may be placed in the lock walls to reduce seepage forces and to lower seepage escape gradients when the lock is dewatered.

Loading conditions that are pertinent to lock floor paving on overburden are (1) lock dewatered with normal lower pool outside of the lock chamber, an unusual loading condition, and (2) lock dewatered with lower pool outside of the lock chamber at the top of the downstream maintenance bulkhead, an extreme maintenance condition. These loading conditions apply to portions of locks located in the lower pool.

Plans 1, 3, and 5—concrete lock floor slabs on overburden. These conceptual plans all have precast concrete paving over all or portions of the lock floors. While analyses have not been provided, the details shown indicate the purpose of the paving, bedding materials, and relief drains, as discussed above. Paving of this type has been constructed at other projects. Two projects have lock floor paving that was constructed under water. All of these have been observed, and their performance can be evaluated. The performance of some has not been entirely satisfactory, and the reason for their poor performance will be discussed. The satisfactory performance of others can also be explained. To understand the reasons for observed performance, it is necessary to understand the principles of seepage control, the importance of good construction practice, and the limits of constructibility. The requirements for good construction practice must be clearly specified and rigidly enforced. The engineering properties of the foundation materials must be fully explored and understood. Analyses must clearly define the seepage paths, seepage quantities under various loading conditions, and the seepage forces. The design of seepage barriers, filter systems, and pressure relief features should reflect each of these factors.

Details of design—Plan 1. Plan 1 is for the extension of a 600-ft lock to 1,200 ft. Details of this plan are described in Chapter 2 and shown in Figures 1 and 2. The principal features of the design of the lock floor paving on overburden are those that affect the quantity and pressure of underseepage from the river to the floor of the dewatered lock. Seepage cutoff walls are driven to a substantial depth (43 ft). Foundation pressure-relief wells are provided to relieve seepage forces when the lock is dewatered. A 12-in. thickness of concrete paving with weep holes is provided. The gradation of the 2-ft-thick bedding under the 12-in.-thick concrete paving is not indicated. Criteria for the design of this type of seepage control and paving system are discussed in the introductory paragraphs of the section "Concrete lock floor slabs on overburden."

Details of design—Plan 3. Plan 3 is for the construction of a new 1,200-ft lock. Details of this plan are described in Chapter 2 and shown in Figures 7-9. Plan 3 has 5- by 7-ft floor beams on 35-ft centers that are designed to carry the full lateral loads, both tension and compression, acting on the lock walls. The design parameters for these beams are discussed below, in the section "Structural concrete lock floor systems." Between the floor beams is a 2-ft-thick concrete lock floor paving, placed on two layers of 2-ft-thick filters, 1 ft of leveling stone, and a geotextile membrane. To limit seepage, a sheet-pile cutoff wall is to be driven to a depth of 42 ft. Criteria for the design of this type of seepage control and paving system are discussed in the introductory paragraphs of the section "Concrete lock floor slabs on overburden."

Details of design—Temporary Locks 52 and 53. Temporary Locks 52 and 53 are not designs that are being evaluated in this study. However, it is of some importance that the floors of these locks were paved underwater with 18-in.-thick precast concrete slabs on 2-ft-thick bedding materials. While these slabs have proven to be adequate for scour protection, they have not been tested for the maximum uplift due to seepage. Temporary Locks 52 and 53 were designed with no provisions for dewatering. The economy in designing without provisions for lock chamber dewatering should not be neglected.

Details of design—Plan 5. Plan 5 is for a new 1,200-ft lock. Details of this plan are described in Chapter 2 and shown in Figures 15 and 16. Precast concrete lock floor paving on overburden is used over the downstream 25 percent of the lock. Few details of this paving are shown; however, it may be assumed that the paving is similar to that of Plan 1, discussed above. Foundation relief wells are placed in the sheet-pile cellular lock walls. The cellular walls are founded 30 ft below finished grade. This embedment provides the seepage cutoff that would be used in analyses of seepage. Criteria for seepage control and paving design are discussed in the introductory paragraphs of the section "Concrete lock floor slabs on overburden."

Factors affecting design parameters. Analyses of design are available for none of these projects. Temporary Locks 52 and 53 have no provisions for dewatering. The need to provide for a normal maintenance or an extreme maintenance loading condition was thereby avoided. The lock miter gates are designed so they can be removed and replaced without dewatering the locks. Inspection and maintenance is performed within small areas dewatered using open-ended and open-sided caissons that rest on surfaces to be dewatered. Substantial savings in the cost of locks can be realized if similar provisions can be made to limit areas that may be dewatered.

Plans 1 and 5 have provisions for pumped foundation relief wells to be used during dewatering. Otherwise, the paving designs appear to be minimal because they have only a 2-ft-thick single-stage filter. Graded filters cannot be placed underwater without segregation. The lock floor paving shown for Plan 3 appears more appropriate for underwater construction. This is because Plan 3 has a two-stage filter, a geotextile membrane, and a 1-ft-thick layer of leveling stone. While analyses would be required to substantiate this design for a specific location, it appears to be a superior design to the others. If seepage can be controlled without pumped relief wells, there would be an operational advantage and possibly an economic advantage. Weep holes in the paving are necessary to pass calculated seepage without pressure buildup, which would tend to lift the concrete paving. The required factors of safety against flotation are discussed above. Methods of performing seepage analyses to determine seepage forces, seepage quantities, and escape gradients can be found in EM 1110-2-1901.

Sheet-pile cutoff walls are essential for lock walls founded on piling. The effectiveness of sheet-pile cutoffs driven to depths of 32 to 40 ft will require investigation. Alignment and interlock may be difficult to maintain for these driving depths. The Z-piling has a ball and socket interlock configuration that will not withstand heavy driving. Where pumped relief wells are proposed for

use during lock dewatering, seepage analyses should be made to determine their effectiveness and necessity. Their cost and dependability should be evaluated against the cost and efficiency of multigraded lock floor pressure relief systems, such as the one shown for Plan 3. Seepage control is highly dependent on foundation conditions, and analyses must be site specific. The need for seepage control for locks located downstream of the dam is dependent on the need to dewater the lock. If the need to dewater locks can be eliminated, considerable savings in the cost of locks can be realized. The applicability of factors of safety given in EM 1110-2-1901 for seepage control for embankment dams needs to be evaluated for use with navigation structures. The application of factors of safety against flotation given in ETL 1110-2-307, for structures subjected to uplift under static heads, should be evaluated for applicability for concrete paving subjected to uplift by seepage forces.

Structural concrete lock floor systems

Unlike concrete paving, structural floor systems are designed to resist applied loads that may produce axial stress, shear, and moment. The precast, prestressed lock floor system will be designed by the strength methods of ACI 318 and in accordance with the requirements of EM 1110-2-2104. The ACI guidance for strength design will be acceptable, as long as the load factors and reinforcement percentages given in EM 1110-2-2104 are followed. In addition to the appropriate load factors, a hydraulic factor (H_f) of 1.3 will be applied to all service loads to account for the service requirements of hydraulic structures. For members in direct tension, a hydraulic factor of 1.65 will be used.

Load factors of 1.4 × Dead Load plus 1.7 × Live Load will apply to the floor system where net forces on the floor result in uplift. Load factors of 1.7 × Dead Load plus 1.7 × Live Load will apply to the floor system where net forces on the floor act in a downward direction. Loading conditions for the lock floor include (1) upper pool in the lock chamber with uplift equal to normal lower pool (a normal loading condition), (2) lock dewatered with lower pool uplift acting on the floor (an unusual loading condition), (3) lock dewatered with uplift equal to lower pool at the top of maintenance bulkheads (an extreme loading condition), and (4) construction condition, construction loads due to transport, placement, and ballasting need to be considered.

The hydraulic factor may not need to be applied to construction loading conditions. Normally an increase in stress is allowed for construction loads of short duration. Residual stresses from construction conditions, however, should be taken into account where they are found to be additive to service condition stresses. Dead load of the floor system will apply in all cases. The strength reduction factor, ϕ, will be taken as 0.9 for flexure and flexure with axial tension, 0.70 for axial compression and axial compression with flexure, and 0.85 for shear and torsion. Loads on the structural lock floor consist of the various pool levels in the lock chamber, the dead weight of the floor system including any ballast applied, and uplift from the river adjacent to the lock walls.

Depending on the connection of the floor system to the lock walls, lateral tension or compression, shear, and moment may be induced into the floor system. Where these forces are transferred between lock walls and floor systems, adequately designed connections must be made in accordance with provisions of ACI 318, subject to the requirements of EM 1110-2-2104. For Plan 4, corrugations are provided in the precast lock wall units against which cast-in-place concrete in the lock floor will be placed. Where landing pads supported on bearing piles are used, they must be capable of supporting the weight of the precast lock floor units placed on them. Their capacity should be in accordance with EM 1110-2-2906, "Design of pile foundations." Pile capacities should be verified by load testing of prototype piles.

Plans 2 and 4—structural concrete lock floor systems. These conceptual lock plans have precast, prestressed, structural lock floor systems. They may be lifted or floated in, then ballasted into place, and attached to the lock walls and/or piles underwater. The structural floor members are designed to resist loads from the lock walls and may improve lock wall stability.

Details of design—Plan 2. Plan 2 is for the extension of an existing 600-ft lock that is founded on piling. This plan is described in Chapter 2 and shown in Figures 4 and 5. The lock floor is a structural, cellular, precast, prestressed floor system that is floated into place and ballasted to rest on the footings of the new lock walls. The floor system spans between lock wall footings and carries its own weight, plus the weight of ballast, and the weight of upper pool on the floor. Uplift is subtracted from the other loads. For the purpose of designing bearings, a hydraulic factor of $H_f = 1.3$ will be applied to these loads. Rather than assume a continuous uniform bearing, which may be difficult to achieve, points of bearings will be designated and designed using steel bearing plates and elastomeric bearing pads. An allowable normal bearing pressure of 800 psi is assigned to these bearings by the American Association of State Highway and Transportation Officials' "Standard specifications for highway bridges." The elastomeric bearing pads are attached to the precast floor system at locations of the steel bearing plates. As the pile-supported lock wall footings support the lock floor system, it is important that adequate support be provided, and that the support be uniform to prevent torsional stresses in the lock floor units. The precast lock floor is anchored to the lock wall footings by HP 14 × 117 stub piles that extend from the footings into 2-ft, 2-in.-diam holes formed in the precast lock floor assembly. Details of the connection between the precast floor slab and the lock wall footings are shown in Figure 4.

After the floor system is properly aligned, space around the stub pile is grouted. Spaces between the lock floor system and the vertical face of the lock walls are also grouted to ensure the floor system will act as a strut to carry lateral loads from the lock walls when the lock is dewatered. The HP 14 × 117 stub piles will also be designed to withstand uplift on the lock floor produced when the lock is dewatered. The lock floor system will also be designed to carry lateral tension loads from the lock walls with upper pool in the lock chamber and lower pool in the river. Joints in the precast lock floor system will be provided with gaskets to prevent leakage. Grout is injected under the floor system to fill the void between the precast lock floor and the foundation. This is intended to

prevent any flow of water beneath the floor system, but should not be counted on to provide support for the floor. Lateral tension in the lock floor system is resisted by bearing on the flanges of the HP 14 × 117 stub piles. Ultimate bearing may be taken as

$$P = \phi\,(0.85\,f'_c\,A)$$

where

ϕ = 0.7, a capacity reduction factor

$0.85\,f'_c$ = mobilized compressive strength of grout

A = bearing area of exposed H-pile (Note: flanges of the HP 14 × 117 stub pile may require stiffeners)

T = uplift capacity of the connection under Loading Conditions of Lock Dewatered

Capacity T depends on the bond between grout and the formed hole in the precast floor system. Capacity may also depend on the bond between grout and the HP 14 × 117. Alternately, the later may be achieved by welding shear studs to the H-pile.

Capacity T, based on bond of grout and the inside perimeter of the formed hole, may be expressed as

$$T = \pi\,D\,L\,\mu$$

where

$\pi\,D\,L$ = surface area of the hole having a diameter equal to D and a length equal to L

μ = bond between grout and the formed hole

The Post-Tensioning Institute (1996) suggests bond strengths of 200 to 400 psi. Using corrugated metal pipe as a stay-in-place form, the bond strength can conservatively be estimated as 400 psi.

Loads from the floor system are concentrated at the line of piles under the footing at the face of the lock. These piles carry the full loads from the lock floor system. Lying at the extreme limits of the lock wall pile group, these piles are most significantly influenced by loading conditions that affect the lock walls. Presently, these piles have no lateral restraint under applied vertical load. A final design for these piles will provide some lateral restraint in the form of a shear key. Bearing piles will be designed in accordance with EM 1110-2-2906. Design values are usually confirmed by in situ load tests. Design values are commonly taken as 50 percent of the ultimate test load that results in displacements that are

disproportionate to additional applied loads. Load tests can and should be performed on piles driven in the wet. Tests are to be made on piles driven in both the wet and the dry to determine a correlation that may exist between the two. This information will be needed if tests on piles driven in the dry are to be applied to piles driven in water.

Details of design—Plan 4. Plan 4 is for a new 1,200-ft pile-supported lock. The lock is filled and emptied through five longitudinal ported culverts integrated into the concrete lock floor. Details of this plan are described in Chapter 2 and shown in Figures 10-14. The floor system for this lock consists of precast, prestressed concrete units that are lowered onto prepared landing pads and flat jacks that are supported on piles. These piles are designed to support the weight of the precast lock floor units, the five culvert pipes, and the weight of tremie concrete fill. Prior to placing the tremie concrete, additional piles are to be driven in spud wells within the precast units to maintain alignment and share the water loads that will be imposed on the completed lock floor system. The precast floor and culvert units occupy approximately 70 percent of the lock floor. The lock floor system will be completed by placing tremie concrete between the precast floor and culvert units and the lock walls. Reinforcing that extends from the precast lock floor units is lapped with reinforcing extending from the precast lock wall shells. With the reinforcing steel and final tremie concrete in place, the lock floor system and lock walls will act integrally as a U-frame lock structure on piles. In Plan 4, as in Plan 2, the limits of structural excavation are well defined by a line of sheetpiling around the perimeter of the lock walls. An 18-in. layer of scour-resistant leveling stone is placed at the base of the excavation. The Z-pile cutoff wall around the perimeter of the excavation is driven through 39 ft of overburden. While grout is injected under the completed floor system and in contact with the foundation, this grout should not be counted on to share loads with the piles that provide more rigid support.

Factors affecting design parameters. Precast, prestressed, structural concrete lock floor members are fabricated under highly controlled conditions, and their design would not be affected by service under water. Loads imposed by construction loading conditions, such as handling, transport, and methods of installation, will need to be accounted for in design. Additionally, any residual stresses that may be additive to service loads will need to be accounted for in design. Otherwise, the hydraulic factor, H_f, for hydraulic service may not apply to construction conditions. This is an issue that will need to be addressed. Pile tests are commonly made in the dry, at the site. Tests can be made on piles driven in water, and the axial load capacities determined can be considered reliable. However, the relationship between the capacity of piles driven in the dry and those driven in the wet is not well established. Testing may be helpful if tests performed on piles driven in the dry are to be applied to piles driven in the wet.

The connection of structural concrete lock floor members to lock walls by placement of tremie concrete does not appear to be unduly complex. However, inspections by a diver should be made to ensure that the area to be filled is initially free of foreign material and is completely filled with tremie concrete after the connection is made. Selection of a suitable mix and methods of delivery will need to be given special attention. Consideration will need to be given to the

spacing of reinforcing so as not to restrict the flow of tremie concrete. Cover over reinforcing must be within the level of tolerance for finished concrete so as not to leave bars uncovered. This will apply mainly to the top reinforcing bars, since the reinforcing cages will be equipped with spacers on the sides and base of reinforced members. Values of bond strength for grouted connections, development length for reinforcing, and lapped splice lengths required for high-performance concrete are taken from available design guides. These may require verification by investigative testing. The depth to which sheetpiling used as cutoff walls can be driven intact, or may need to be driven, for effective seepage control should be further evaluated.

Concrete-paved lock floors with tension and compression lock wall ties

Locks founded on piles have paved floors, either cast-in-place or precast slabs, that are lifted and set in place. These locks may also have structural members, as shown in Plan 3. These members are interspersed with the lock floor paving and serve as compressive struts to resist lateral loads on the lock walls when the lock is dewatered. These same members serve as tension ties to resist lateral loads on the lock walls when the lock is filled to upper pool level and the lock is located in a lower pool. Precast paving slabs and filter bedding materials placed between structural floor beams serve only as lock floor paving, and are designed in the same manner as described above in the section "Concrete lock floor slabs on overburden." The concrete paving between the floor beams may be placed using self-leveling tremie concrete or by screeded tremie concrete. The design of the structural members that serve as compression struts or tension ties is the same as described above for the "Structural concrete lock floor system." Bearing piles used for support of the floor beams are designed in accordance with EM 1110-2-2906.

Details of design—Plan 3. Plan 3 is for a new pile-supported 1,200-ft lock located riverward of an existing 600-ft lock. Details of this plan are described in Chapter 2 and shown in Figures 7-9. The main structural members of the lock floor are 5- by 7-ft floor beams, which act as tension ties and compression struts for the lock walls. The floor beams are significantly stiffer than the lateral resistance offered by the vertical lock wall bearing piles. The floor beams are, therefore, assumed to carry all of the lateral loads on the lock walls and to thereby improve the stability of the lock walls. The floor beams themselves are supported on bearing piles that are equipped with bearing pads to provide uniform vertical support for the floor beams. Additional piles are to be driven through spud wells to provide additional support. The latter will be attached by a grout connection between the piles and the spud wells. Bond between grout and corrugated metal spud well lining is taken as 400 psi, the upper limit of bond recommended by the Post-Tensioning Institute (1996). Bond between grout and pile is taken as 200 psi, the lower limit of the Institute's recommendations.

Factors affecting design parameters. The precast, prestressed floor beams for Plan 3 are structural concrete members that are fabricated, transported, and installed underwater similar to the structural concrete lock floor systems

described above. Factors affecting design parameters will be the same as those described above for the "Structural concrete lock floor system."

Riprap-paved lock floors with bedding on overburden

Criteria for riprap thickness and size, based on velocity and type of flow, have been developed by Creager, Justin, and Hinds (1945) and are used for erosion control of Corps flood control channels. Three feet of quarry-run stone with a 250-lb maximum size stone, placed on 12 in. of bedding, has been used successfully to protect the filling and emptying flumes at Temporary Locks 52 and 53. These flumes are filled and emptied by double 4- by 8-ft sluice gates under low heads of 10 to 12 ft. For use as lock floor paving, the stone and bedding would provide the necessary scour protection against propeller wash and discharge from filling and emptying systems. The stone and bedding material should be designed in accordance with EM 1110-2-1901. A factor of safety of 2.00 against piping of foundation materials due to the development of the critical seepage escape gradient should be provided for the lock dewatered with normal lower pool in the river, an unusual loading condition. A factor safety of 1.5 should be provided with the lock dewatered with the lower pool at the top of the downstream maintenance bulkheads, an extreme maintenance condition. The quarry-run stone may require grout to keep the smaller stone from being dislodged by lock currents.

Details of design—Plan 5. Plan 5 is for the construction of a new 1,200-ft lock located downstream of an existing gated dam. Details of this plan are described in Chapter 2 and shown in Figures 15 and 16. The lock walls are sheet-pile cellular structures, driven 39 ft into overburden. Ported culverts are located in the lock chamber. The lock floor adjacent to the culverts is protected against erosion and foundation piping by a 6-ft-thick layer of riprap, which is placed on 5.5 ft of filter material. Foundation relief wells are placed in the cellular lock walls to be pumped when the lock chamber is dewatered. The depth and width of the cellular sheet-pile lock walls significantly extends the seep path and helps control underseepage. The requirements for underseepage control are as given in the preceding paragraph.

Factors affecting design parameters. The sheet-pile cellular walls are driven to a depth of 39 ft. The capability of driving sheetpiling to this depth without severe damage to the piling and the need to achieve this penetration for seepage control need to be addressed. Design of the pile-supported culverts needs to be reviewed. It is not clear how a positive connection will be made between the piles and the culverts. Piles with suitably equipped landing pads and leveling flat jacks will need to be designed to resist the dead load of the culverts and water. Additional piles on each side of each culvert may be needed to provide lateral support, with suitable attachments to provide additional vertical support. Seepage analyses should be made to size the riprap and filter materials. While the plan is conceptual, site-specific foundation information will be required for a detailed seepage analysis. The necessity for the foundation relief wells, and the required sheet-pile lock wall penetration needed to control seepage, will need to be evaluated. Factors of safety against the critical escape gradient are taken from

EM 1110-2-1901, for embankment dams. Sensitivity analyses should be made to determine if these factors of safety are sufficient for seepage forces associated with navigation lock floors constructed underwater.

Lock Culverts

When locks are constructed in the dry, it is economical and convenient to locate culverts within the lock walls. When locks are constructed in the wet, it is often cost effective to place the culverts in the lock chamber or behind the lock walls. With the culverts located outside the lock walls, it is often expedient (schedule-wise) to install the culverts in the wet. Where culverts are located within the lock chamber, it is important that they be founded on competent rock or on piles, to be stable under the water forces to which they will be subjected. Filling and emptying flumes located outside the lock walls have been found to operate satisfactorily and to be an economical substitute for culverts at some projects. Culverts located behind lock walls may be accessible during construction and after project completion. Advantages of this location will be addressed under a modified Plan 2. Culverts will be designed in accordance with ACI 318, as modified by EM 1110-2-2104. Piling used to support lock culverts will be designed in accordance with EM 1110-2-2906.

Lock extensions with no new culverts

Locks may be extended without additions to the filling and emptying systems. Using only the original filling and emptying system, additional time is required to fill and empty the extended lock chamber. Operation of the valves must be slowed to reduce hawser pulls to acceptable limits as water flows from one half of the lock chamber to the other. Where the lengthened filling and emptying time is found to be acceptable, substantial savings in construction time and costs can be achieved.

Details of design—Plan 1. Plan 1 is for the extension of a 600-ft lock to 1,200 ft. Details of this design are described in Chapter 2 and shown in Figures 1-3. The lock is to be extended with no revisions to the existing filling and emptying system. No new culverts will be required.

Factors affecting design parameters. As culvert additions are not used, there are no affected design parameters.

Culverts located in back of lock walls

The culverts and conduits used to fill and empty the lock chambers are exposed to (1) normal operating conditions with upper and lower pools in the lock chamber and culverts; (2) unusual loading conditions, with the lock chamber and culverts dewatered at normal pools, and construction loading conditions; and (3) extreme loading conditions, with lock chamber and culverts dewatered, with pools to top of maintenance bulkheads. Buoyancy of culverts is a condition that

requires close evaluation. Analyses of buoyancy calculated in accordance with ETL 1110-2-307 should be applied to the culverts. Factors of safety required are FS $\geq$ 1.50 for normal loading conditions, FS $\geq$ 1.30 for unusual loading conditions, and FS $\geq$ 1.10 for extreme maintenance loading conditions. Culverts and conduits should be designed in accordance with ACI 318, as modified by EM 1110-2-2104. Culverts and conduits should have sufficient thickness, such that flush-mounted tongue-and-groove joints with pressure-type gasket fittings can be used. Thrust blocks are provided at bends in conduits.

To provide the required factor of safety against flotation, excavations in rock for conduits will be back-filled with tremie concrete. The culverts and conduits will be designed to be dewatered at normal lower pool (an unusual loading condition) and with the pool at the top of lock maintenance bulkheads (an extreme loading condition). The conduits will also be designed for internal pressures due to pressure gradients that develop upon lock filling and emptying. Installation underwater will subject the culverts and conduits to construction loads that may be difficult for the designer to predict. The specified load factors from EM 1110-2-2104 will apply to all loading conditions, as will the required hydraulic factors (with the exception of that for the construction condition, which will need to be further evaluated). Submittals may be required of the contractor to evaluate stresses induced by construction practices. This will be addressed in the section below, titled "Design parameter issues."

Plans 2 (Alternate), 6, and 7. These plans have culverts or conduits that are located either in back of the lock walls or (for Plan 7) twin conduits located in the downstream lock approach.

Details of design—Plan 2 (Alternate). The alternate lock wall units are precast, prestressed thin-wall sections stiffened by counterforts. The culverts rest on the heel of the lock walls and pass through the counterforts. This plan is described in Chapter 2 and is shown in Figure 6. The lock floor system will be the same as used for Plan 2. The culvert may be exposed in back of the walls or may be covered by fill. After installation and alignment, a designed structural anchorage will be provided.

Details of design—Plan 6. Plan 6 is for a new 1,200-ft lock to be constructed landward of an existing 600-ft lock. This plan is described in Chapter 2 and is shown in Figure 17. Plan 7 has twin 15-ft, 6-in.-diam conduits that exit from the new lock and cross the downstream lock approaches to discharge on the riverside of the riverward lock. The discharge conduits will be placed in an underwater excavation in rock. Landing pads or prepared cradles will be used to position the conduit segments, which will be joined together either underwater or prior to being set in place. To ensure that proper alignment is maintained and flotation will be prevented, voids under, around, and over the conduits will be filled with tremie concrete while the conduits are held in position. An outlet manifold will be located riverward of the river lock's lower guard wall. Bulkheads will be provided to allow the conduits to be dewatered for inspection at normal lower pool, or with lower pool at the top of downstream maintenance bulkheads. These loads will be considered unusual and extreme maintenance loading conditions. Normal loading conditions considering pressure

surges due to filling and emptying the lock will be analyzed as a usual loading condition.

Details of design—Plan 7. Plan 7 is for a 600-ft extension of an existing 600-ft landward lock. This plan is described in Chapter 2 and is shown in Figure 18. The 16-ft, 6-in.-diam intake conduit for the 600-ft lock extension will be constructed within a braced excavation, an open-cut excavation, or by a tunneling process at depths between 35 and 50 ft. Criteria to be followed in these designs will depend on the method chosen, but in general will consider all loads to which the conduits will be subjected.

Factors affecting design parameters. Alignment of conduits and culverts should be fairly precise, and it will be necessary to maintain alignment under pressure of tremie concrete that may tend to dislodge them. Systems of anchored cradles will need to be devised to achieve the necessary alignment during the construction process. Conduits and culverts will require pressure tests prior to final installation. The precast conduits and culverts will be fabricated under ideal construction conditions. Although the culverts and conduits will in many cases be inaccessible, the prescribed load factors should be adequate. The hydraulic factor, H_f, is considered appropriate for service conditions, but may not be required to limit stresses during construction. This is an issue listed for further evaluation.

Culverts in lock chamber

Plans 4 and 5 are for new 1,200-ft locks. Plan 4 has bearing pile-supported concrete lock walls. Plan 5 has sheet-pile cellular lock walls. Both Plans 4 and 5 have culvert intakes located in the upper miter gate sills. The sill also contains a mixing chamber that both fills and empties the lock chambers. Intakes are controlled by butterfly valves in the upstream side of the miter gate sills. Discharge gates are located in the ends of the mixing chamber. Longitudinal ported culverts extend approximately 75 percent of the length of the lock chambers. Sheet piles are driven to depths of 39 ft, and pumped foundation pressure relief wells are provided in the sheet-pile cellular walls to reduce underseepage with the lock dewatered. EM 1110-2-2906 will be used for the design of bearing piles used in the design of Plans 4 and 5. This includes the piles supporting the landing pads that support the dead load of the precast lock culverts and lock floor systems.

The culverts in Plan 5 may require addition piles driven on each side of the culverts for lateral support and additional vertical support for both bearing and uplift. Connections will be designed for the required loads. Tremie concrete will be placed in voids between the sheet-pile cellular walls and the culverts to provide additional stability to the culverts and to bring those areas to lock floor grade. Flotation analyses will need to be made for the culverts in Plan 5, in accordance with ETL 1110-2-307. Piles sufficiently attached to the culverts may be taken into account in the analyses. Seepage forces as well as hydrostatic pressures should be considered when considering the dewatered condition.

Details of design—Plan 4. Plan 4 is a conceptual plan for a new 1,200-ft lock. A large part of the structural concrete lock floor is precast, prestressed and has compartments for five longitudinal 8-ft-diam ported steel culvert pipes. This plan is described in Chapter 2 and is shown in Figures 10-14. Details of design will be as described in this section's introductory paragraphs above. Construction details will be as described in Chapter 4.

Details of design—Plan 5. Plan 5 is for the construction of a new 1,200-ft lock located downstream of an existing gated dam. Details of this plan are described in Chapter 2 and shown in Figures 15 and 16. The lock walls are sheet-pile cellular structures. Foundation relief wells are placed in the lock wall cells to be pumped when the lock chamber is dewatered. Longitudinal culverts with ports are located in the lock chamber. Details of design will be as described in this section's introductory paragraphs above. Construction details will be as described in Chapter 4.

Factors affecting design parameters. The precast structural concrete elements and steel pipe are fabricated under ideal conditions and should serve underwater with no problem. Construction loads due to handling, transport, and placement will need further investigation, but may not require the use of a hydraulic factor ($H_f = 1.3$) for hydraulic service. This should be given consideration before reaching a final decision. Design in accordance with appropriate Engineer Manuals, Engineer Technical Letters, and ACI 318 should be adequate. Seepage analyses and evaluations should be the basis for determining the actual depth of sheet-pile seepage cutoffs.

Design Parameter Issues

Individual panels/assemblies

As applied to concrete lock floor paving, suggestions have been made that articulated mattresses be considered instead of other forms of paving. Articulated mattresses have been used successfully to control the erosion of the banks of rivers, and large areas have been protected in this manner. Specialized equipment has been developed to expedite placement. Questions that need to be addressed are these: (1) Can the specialized equipment that is available operate within the confines of a lock chamber? (2) If not, can equipment be developed and its cost amortized? (3) If interlocking precast slabs are preferred, can the slabs be individually placed and interlocked with mechanical fittings? and (4) Can interlocking configurations, such as used in driveway construction, be beneficial?

It may be worthwhile to explore these options. The system of small individual paving blocks placed in running bond, with 2-in. joints, and shallow lock floor submergence, has proven successful at Temporary Locks 52 and 53. This may prove to be the least costly alternative, and has the least risk of construction problems.

Flexural strength

The flexural strength of paving slabs, either with precast units or slabs placed by tremie methods, would not appear to be critical to performance. The issue to be resolved is, Should paving be expected to span sizable variations in foundation grade? It would appear sufficient for paving to be held together by reinforcing steel in the event damaging cracks do occur. Reinforcing in the amount of 0.2 percent should be adequate for this purpose. Precast slabs must have sufficient reinforcing steel to be lifted with a four-point pickup assembly, without danger of breaking under construction conditions. The requirements of the ACI Building Code and EM 1110-2-2104 should be adequate for the flexural strength of lock culverts and structural concrete lock floor systems.

Shear strength

The shear strength of concrete of lock floor paving should not be a design issue. The shear strength of culvert members, structural concrete lock members, and structural systems may be adequately designed using the requirements of the ACI 318 Building Code Requirements and EM 1110-2-2104.

Consideration of load types

Load types for culverts and slabs are usual, unusual, and extreme. Construction loading is normally taken as an unusual loading condition. The allowable factors of safety are as previously described. Loads imposed on culverts and paving are described in EM 1110-2-2906. Loads that act under normal pools and under normal operating conditions are considered usual loading conditions. Loads that occur as a result of lock dewatering at normal pools are considered unusual loading conditions. These include seepage forces as well as hydrostatically applied water loads. Loads that result from infrequent events, such as passing ice and debris by skimming, or loads resulting from use of the lock as a floodway, are considered extreme loading conditions.

Span/thickness ratio

Span/thickness Ratio would apply only to structural concrete lock floor members, floor systems, and culverts. It would appear that these would only be required to meet the minimum requirements of the ACI Building Code and EM 1110-2-2104. Load capacity may dictate deeper cross sections. Otherwise, there would not appear to be a need for special requirements for precast, prestressed concrete lock floor systems or for culverts.

Strength reduction factors

Strength reduction factors are meant to apply to uncertainties of fabrication or behavior under loads. For precast, prestressed concrete lock floor systems, there

would appear to be little uncertainty, as the members are fabricated under a highly controlled environment. For structural concrete lock floor members constructed underwater, there may be uncertainties in fabrication and properties of special materials placed underwater. For these, it would appear that strength-reduction factors would need to be addressed for specific cases. Any additional reduction in strength factors should be based on uncertainties of underwater construction methods and limited knowledge of special materials used in the work. These may be project or site specific, or based on demonstrated capabilities of individual contractors.

Load factors

Load factors are intended to apply to the uncertainty of loads for which structures are to be designed. Live and dead loads for hydraulic structures are reasonably well defined, with the exception of hydrodynamic loads. As some structural elements are to be installed underwater, the methods may not be entirely known by the designer. It is, therefore, considered advisable to evaluate these load factors individually based on the uncertainties of the particular loads and the susceptibility of the structure. Sensitivity analyses are considered beneficial in determining the probability of design loads being exceeded.

Hydraulic load factor

These load factors are intended to apply to hydraulic structures to account for severe service requirements. A hydraulic load factor ($H_f = 1.3$) is normally applied in addition to the customary load factors, except for members in direct tension, where a factor of 1.65 is applied. It has been suggested that the hydraulic load factor need not be applied to construction conditions. Commonly, allowable stresses are relaxed substantially for erection stresses of short duration. Primary concerns are for safety during construction. Structures must be put in place without damage, and residual stresses induced by installation practices must be accounted for in the completed structure. Alternatives are to specify erection methods and assign a hydraulic factor that is appropriate for that method, or to require of the contractor an erection plan, including analyses to show design goals are met. The construction loading condition does need to be addressed to ensure a satisfactory end product.

Concrete cover

The concrete cover specified by EM 1110-2-2104 would be adequate if it could be ensured in structures constructed underwater. To arbitrarily increase concrete cover would not ensure that it, too, would not be met. This problem needs to be addressed. However, it may be best to specify required cover and require the contractor to submit a quality control plan that addresses methods of achieving the desired concrete cover.

Crack width criteria

With the limitations placed on the strength of reinforcing and the percentage of reinforcing by EM 1110-2-2104, cracking investigations are not required for service loads. Generally, concrete placed by tremie methods will exceed the normal design strengths in order to achieve satisfactory placement. High-performance concrete will likely be used for its self-leveling characteristics. Unless research indicates that high-performance concrete is more prone to crack than regular tremie concrete, cracking should not be a problem for the structures involved. Likewise, precast, prestressed concrete members should not be a problem.

Concrete strength

Concrete strength is normally taken from design and load capacity requirements. Generally, concrete with a 30-day compressive strength (f'_c = 5,000 psi) is used for precast, prestressed concrete; a value of f'_c = 4,000 psi is used for precast concrete; and f'_c = 3,000 psi is used for conventional reinforced concrete. For precast culverts and conduits, concrete with a 30-day compressive strength of 5,000 psi will be used, for its needed strength for handling and greater load capacity. These should be sufficient for the structures involved in this study.

Reinforcement limits

There do not appear to be any special requirements for this study that would override the requirements of EM 1110-2-2104 or necessitate more stringent requirements.

ASTM materials specifications

Consideration will need to be given to materials used for underwater construction. Where underwater construction dictates the choice of materials, the need should be justified. Otherwise, commonly used materials chosen from options in applicable Corps of Engineers' guidance for lock construction should be specified.

Reinforcing steel. EM 1110-2-2104 strongly recommends American Society for Testing and Materials (ASTM) A 615 (Billet Steel), Grade 60, for reinforcing steel. There does not appear to be any compelling reason to deviate from this guidance. If for any reason corrosion appears to be a problem, epoxy-coated reinforcing would be an option.

Prestressing tendons. For precast, prestressed, or post-tensioned members, ASTM A 416, Grade 270K, 0.6-in.-diam, epoxy-coated, low-relaxation, 7-wire strand, is the most commonly used material in the industry today. This material should serve well for underwater service

Post-tensioning bars. Where post-tensioning bars are used for construction purposes or as a part of the permanent work, ASTM A 722, Grade 160, epoxy-coated bars should serve well underwater.

Structural steel. Where embedded and temporarily exposed structural steel is used in underwater connections, ASTM A 588 and A 242 have enhanced weathering characteristics under atmospheric exposure. Underwater, they provide only slightly enhanced corrosion resistance. They do provide higher working stress levels. Structural steels used in underwater connections are expected to bond to concrete even though mechanical anchorages may be required. For this reason, these structural steels should not be painted. A mild coating of rust will be beneficial to the development of bond stress. For these reasons, either ASTM A 588 or A 242 should be the steel of choice. For steel H-piles and sheet piles, ASTM A 572 steel should be used unless site conditions and design dictate otherwise.

Reinforcing splices/couplers

Generally it is more economical to lap splice reinforcing steel No. 11 and under. It may be advisable to limit bar sizes to No. 11 and under, because of the difficulty of coupling larger bars underwater. Thermit welded couplings have superior strength and have minimal relaxation; however, an exothermic process cannot be used underwater. Required development and splice lengths made in high-performance concrete may require laboratory testing to verify design, where bond lengths are not well defined.

Deflection control

With the limitations placed on the strength of reinforcing and on the percentage of reinforcing by EM 1110-2-2104, deflection investigations are not required for service loads. Generally, concrete placed by tremie methods will exceed the normal design strengths in order to achieve satisfactory placement. High-performance concrete will likely be used for its self-leveling characteristics. Unless research indicates that high-performance concrete will deflect more than regular tremie concrete, deflections should not be a problem for the structures involved. Likewise, precast, prestressed concrete members should not be a problem.

Lock chamber dewatering capability

Analytical evaluations need to be made to establish the added costs and benefits of designing locks that may be dewatered. All locks have not been designed to permit dewatering. Temporary Locks 52 and 53 are examples. Despite this limitation, significant repairs have been made to underwater metals using

small caissons and skin cofferdams with gaskets that fit the structure configuration, which allow repairs to be made in the dry. With advanced planning for maintenance, significant costs of construction may be saved without detracting from the ability to maintain the structure. Provisions for localized dewatering, in lieu of lock chamber dewatering, should be evaluated.

4 Construction Methods

Construction of Navigation Structures

The Corps of Engineers has been constructing locks and dams for the Nation's waterways for nearly a century. This history has provided a valuable resource of experience as new project construction is undertaken. Lessons learned have been passed on from one project to the next, resulting in better construction practices. However, with the advent of innovative construction of navigation locks and dams, much of the Corps of Engineers' experience in the construction of these structures provides little information that is relevant to the construction practices needed for this type of construction. The construction practice has gone from construction in the dry using conventional construction procedures to construction in the wet, which requires the use of construction practices of other industries as well as the development of new practices. The conventional methods for construction provided a great deal of quality control through sight alone, whereas the innovative methods often require special means for quality control. In-the-wet construction will require investigation and development to enable the Corps to continue to construct navigation structures that are safe and functional.

Conventional construction

In the past, the Corps has constructed locks and dams on the inland waterways using a cofferdam system that typically consisted of sheet-pile cells and a deep-well dewatering system. This often required several cofferdam structures to be built so as not to constrict the flow of the river too much. Often, the cofferdam area had to be much larger than the area of the structure being built to allow room for stability berms on the inside of the cofferdam.

Upon completion of the cofferdam and implementation of the dewatering system, work can be performed in the dry. For soil- and pile-founded structures, this includes grading of the foundation, driving of sheet-pile cutoffs, placement of mud mats, and driving of piling as needed. Core borings may be taken for structures founded on rock to establish the depth of rock excavation needed. Once the foundation is prepared, formwork is erected, reinforcement is placed, and concrete is placed in the formwork. Often a conveyor system is used for placing the concrete to provide continuous placement. As the concrete is placed,

vibration is used so that the concrete is properly consolidated. As a lift of concrete is completed, a finishing surface is put on the top of the lift, and various methods of post-placement care are used—such as insulating blankets in the winter and damp burlap in the summer. Any embedded metals that may be included in the structure are added prior to the placement of the concrete. If any post-tensioning is to be done, it is also performed in the dry.

During the construction process, quality control is exercised through several means. Conventional surveying is used for a number of quality-control checks. Surveying is used to place the cofferdam cells, establish proper grades, and to place cutoff walls, piles, embedded metals, and forms. Tape measurements are used to supplement the surveying measurements. In addition, visual inspections of items such as forms, reinforcement placement, and finished concrete can easily be made.

Innovative construction

Construction using innovative methods provides a number of challenges for the Corps of Engineers. One of the ways to meet these challenges is to use construction techniques and methods from other industries, primarily the offshore industry, and apply them to the construction of navigation projects. While making use of practices from other areas will not always provide the exact approach that may be needed on a navigation project, it certainly provides a good starting point, in many cases. In the sections below, various methods that have been used for underwater construction will be discussed. These discussions will be followed by descriptions of how some of these construction methods may be used for constructing lock floor slabs and culverts underwater.

Foundation Construction

As for any construction project, the first item of business is to prepare the foundation. This includes properly grading the foundation, stabilizing it, and improving its capacity. For construction in the dry, there may be some maintenance after the foundation has been prepared, but for underwater construction, particularly in a flowing river, it will be an important and vital aspect.

Underwater foundation preparation

Removing or leveling the area of construction will typically be the first order of business. Often, removal of materials and leveling can be done with some traditional methods, if the foundation does not consist of rock. (Rock excavation is discussed below.) One method that can be used to remove materials is a clamshell bucket. Another approach to remove materials is to use a conventional dredge. Typically though, while these methods are effective for removing materials, they will likely not leave the foundation graded to the extent that will be needed for placing a structure.

To achieve leveling, a heavy girder could be suspended from a barge, and leveling could be accomplished by dragging the beam across the area in need of leveling. It is important in this technique that the beam be heavy enough to ensure that it levels the material and does not rise up over hills on the riverbed. A more sophisticated approach is to build a screed. A large frame is constructed that is put on the river floor. Jacks are used to level the frame, and then the screed within the frame is drug along the riverbed. The general concept of a screed frame is shown in Figure 19 (Gerwick 1999).

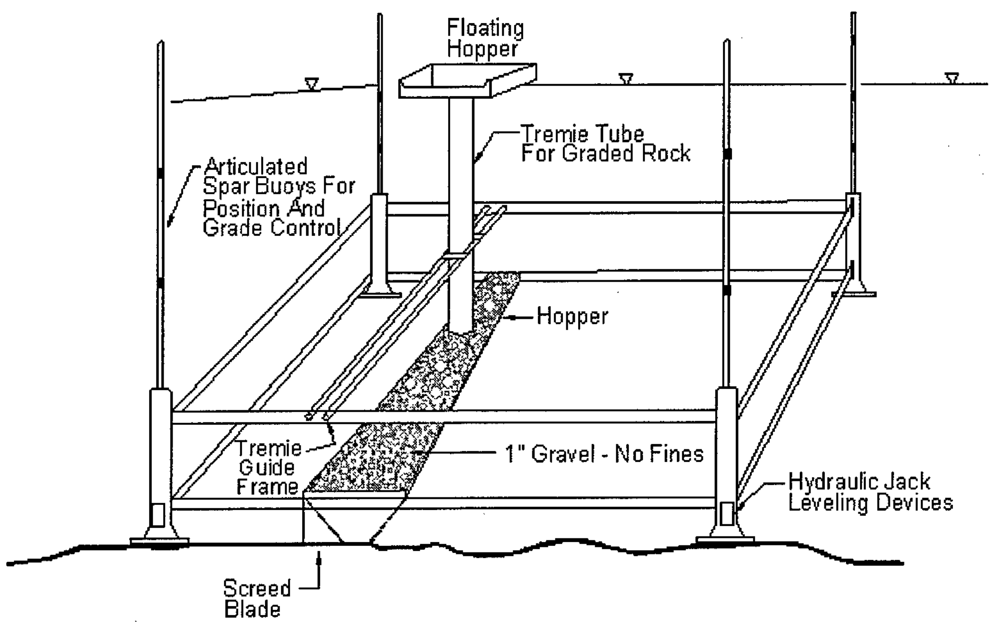

Figure 19. View of screed frame

Sometimes boulders are contained within softer materials on the floor of the river. This can create problems when trying to level the surface. Small boulders (18 in. or less) can typically be moved with the methods discussed above (i.e. clamshell, screeding) but for larger boulders, special measures may be needed. One method is to attach cables to the boulders and drag them out of the construction area with tugs. Another method is to place shaped charges on the boulders to shatter the boulders into smaller, manageable pieces. High-pressure jets have also been used to break up large boulders into smaller pieces (Gerwick 1999).

Rock excavation

Excavating rock to a prescribed elevation underwater appears to be a very daunting task. However, there has been success in excavating rock underwater. Depending on the formation of the rock, different methods may be effective for

excavation. In addition, as with softer materials, methods for removing rock may not be the same methods that are needed to level the rock.

For stratified rock, large hydraulic backhoes mounted on a barge may be capable of pulling up sections of the rock. This has been accomplished in waters over 50 ft deep. Explosives have often been used for breaking up of foundations for removal. A particularly effective method is to drill holes at regular spacing and place charges in the holes. To contain the extent of the blasting, line drilling may be employed to form a weakened boundary through which the fractures (due to the explosions) will not extend (Gerwick 1999). Line drilling using a template can be an effective way of methodically blasting areas that need to be modified (Duncan 1972). With any blasting operation, the fractured rock that results must be removed, typically through conventional means.

High-pressure water can also be used to propagate fractures in the rock for removal. This method has been used in quarry operations in the dry, but it is believed that it could be extended into underwater applications. There is also the use of rock-breaking chisels that are driven by hydraulic or vibratory hammers and are available for use underwater (Gerwick 1999). As with explosives, the chisel is used to break the rock in portions that can be removed using conventional methods such as clamshells.

As was the case for soft foundation materials, the methods used to remove the rock are not necessarily the methods used for leveling the rock. Grinding can be used to achieve leveling of the rock. There are machines with rotating bits that move horizontally along the surface but require a structural frame to hold the mechanism rigidly in place. A machine that has grinding wheels moving in the vertical direction (axis is horizontal) is more effective. Rock-trenching wheels like this were used for a cable crossing across the English Channel. A large-diameter rotary drill mounted on a frame making vertical holes in the rock in a regular pattern, as shown in Figure 20, could be a method for breaking the rock and, with close control of the drill, for obtaining a level surface (Gerwick 1999).

Fill for underwater placement

There may be some cases in which a foundation needs to be built up using foundation materials instead of more costly structural materials. For these cases, graded rock is often an option that can be considered; however, selecting the proper gradation is critical to ensuring the stability for this type of foundation. Using such a rock-filled mattress has been shown to be viable, as demonstrated on the Fairview Terminal in Prince Rupert, British Columbia (McCammon and Fitzell 1991). For the Fairview Terminal it was shown that the rock-filled mattress could provide the foundation needed by using finer, better graded materials, flattening the mattress front fill slope, providing scour protection, and performing in situ densification using vibrocompaction. Other compaction possibilities include a vibratory plate compactor to be used on the surface, roller compaction through the use of a remote-controlled underwater bulldozer, a pile or vibration hammer, and the repeated raising and dropping of a heavy weight (Gerwick 1999).

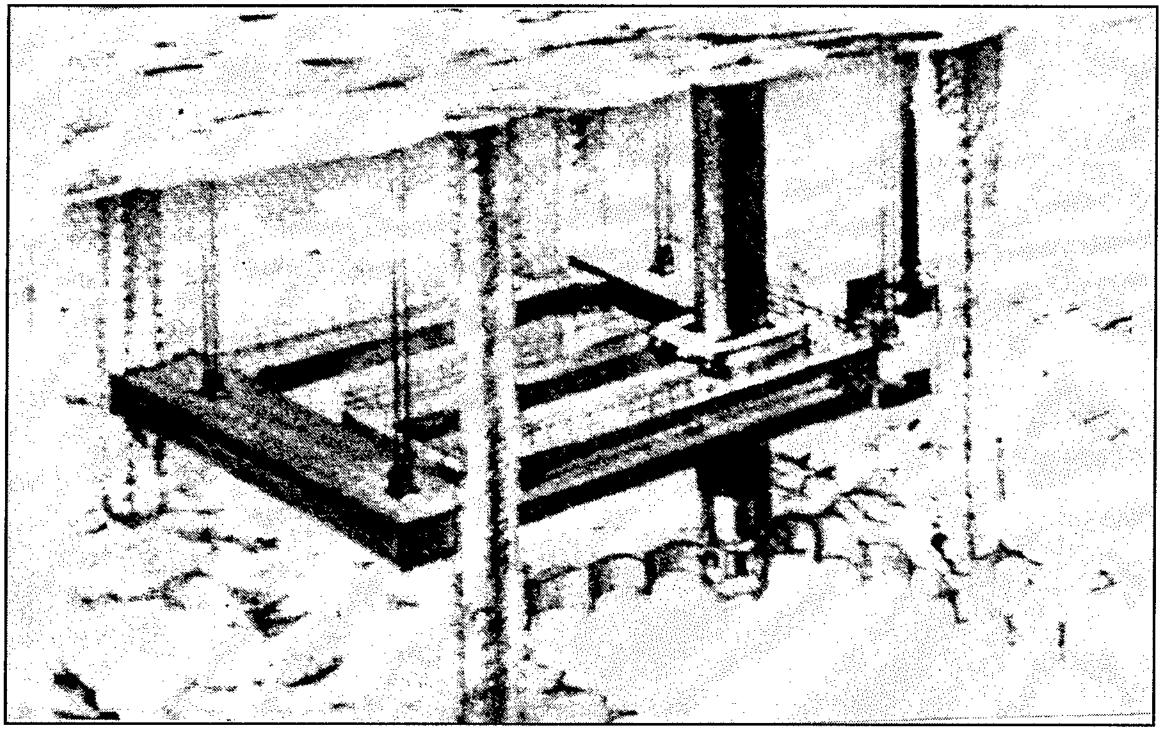

Figure 20. Frame for rotary drill used in pattern drilling

Increasing foundation capacity

For structures founded on soils it is generally necessary to somehow increase the foundation support to provide the necessary bearing capacity of the structure. Different support conditions call for different methods of providing the needed bearing capacity. Available methods include the use of steel H-piles, steel pipe piles, concrete piles (pre- and post-tensioned), caissons, minipiles, and soil mixing. These different methods for increasing soil capacity will be discussed below.

Steel H-piles and pipe piles have been used in a number of civil works projects over the last several decades. Although most projects using steel piles have been built in the dry, equipment does exist for driving these types of piling in the wet. This is most recently evidenced by the pile test done by the Corps' New Orleans District using underwater pile-driving equipment.

Caissons are also certainly a viable means for increasing foundation capacity. The Braddock Dam on the Monongahela River is in the construction phase and will be making use of caissons as its foundation support. The ability to construct caissons underwater in a river environment has been demonstrated by test caissons that were constructed and tested in August 1998.

Minipiles are another type of support that can be used. Minipiles are usually less than 12 in. in diameter and are constructed similarly to a caisson, in that they are installed by drilling and grouting. A minipile develops very good frictional resistance, which allows it to achieve large load-carrying capability in very short distances. Minipiles are constructed using high-strength, flush-jointed steel pipe for drill casing that can be withdrawn or left in place for additional capacity. Reinforcing can also be included in minipiles with a partial steel casing, as shown in Figure 21 (Gaddie in USACE 1997). Minipiles can be a feasible option for structures that do not require a tremendous increase in bearing capacity.

Soil mixing is a technique that involves drilling into the existing foundation using an auger and mixing paddles and injecting a cement/water grout so that it mixes with the soil. This creates a soil-cement that has a compressive strength of 100 to 200 psi. As part of the Third Harbor Tunnel in Boston, MA, soil mixing was used in conjunction with soldier piles (W21 Grade 50 steel beam) that were placed in some of the holes drilled by the auger to further increase the soil capacity (Pearlman and Himick 1993). Implementation of these methods for use in underwater applications appears to be viable and appealing for some instances where load capacities may not be high.

Maintaining foundation preparation

Once the foundation has been prepared and is ready for the structure to be placed, the maintenance of the foundation becomes of primary importance. Construction in the dry presents few problems of foundation maintenance; however, when construction is underwater, in water that is flowing, major problems can arise.

The biggest problem with constructing structures in a river environment, particularly the environments of the Ohio and Mississippi Rivers, is the large silt load contained in the water. Basically, two methods exist for maintaining a foundation that has been prepared. The first is to protect the prepared area with shields that cause the silt to pass over the area that needs to be kept clean. The other is to place an erosion-resistant material on the prepared foundation immediately after the preparation is complete, to have a more permanent reference point to excavate to, just prior to beginning construction of the structure. This could include items such as a thick filter cloth or a layer of tremie concrete. Filter cloths can be reinforced with heavier materials, including stainless steel (Gerwick 1999).

Another possibility for maintaining excavations would be to use articulated mats, similar to those used on the lower Mississippi River to reduce erosion of the banks. Figure 22 shows the mat-sinking process used on the Mississippi River. Mattresses with voids that can be filled with sand, stone, or grout are also a possible means of maintaining underwater excavations.

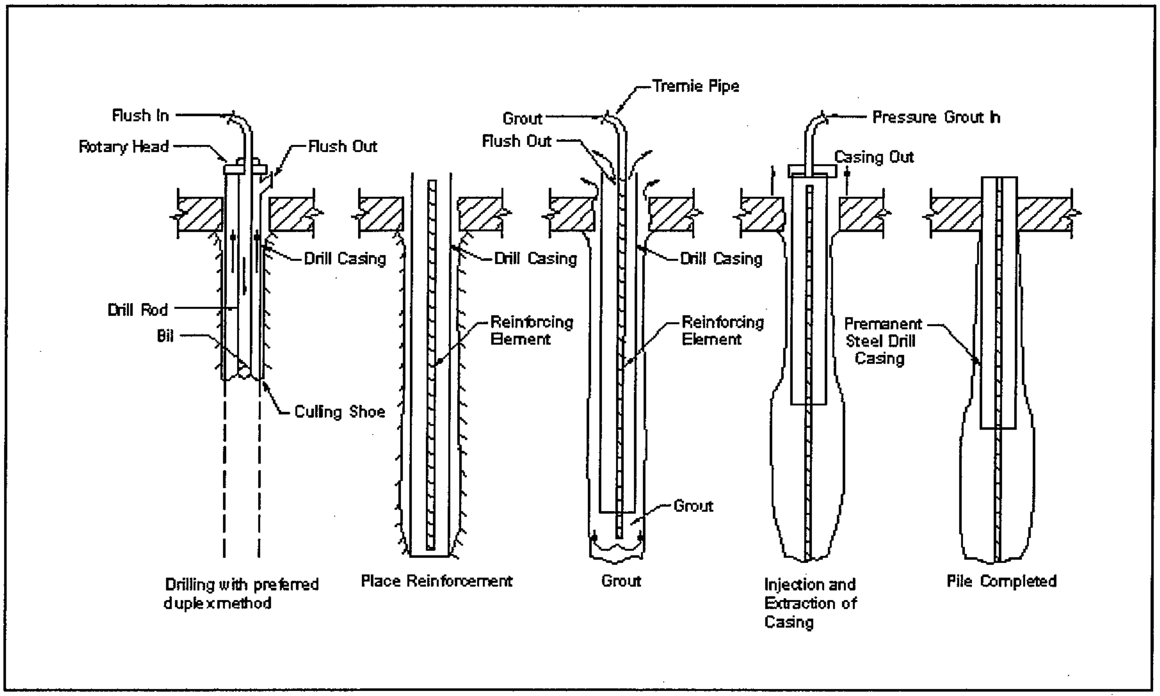

Figure 21. Details of minipiles

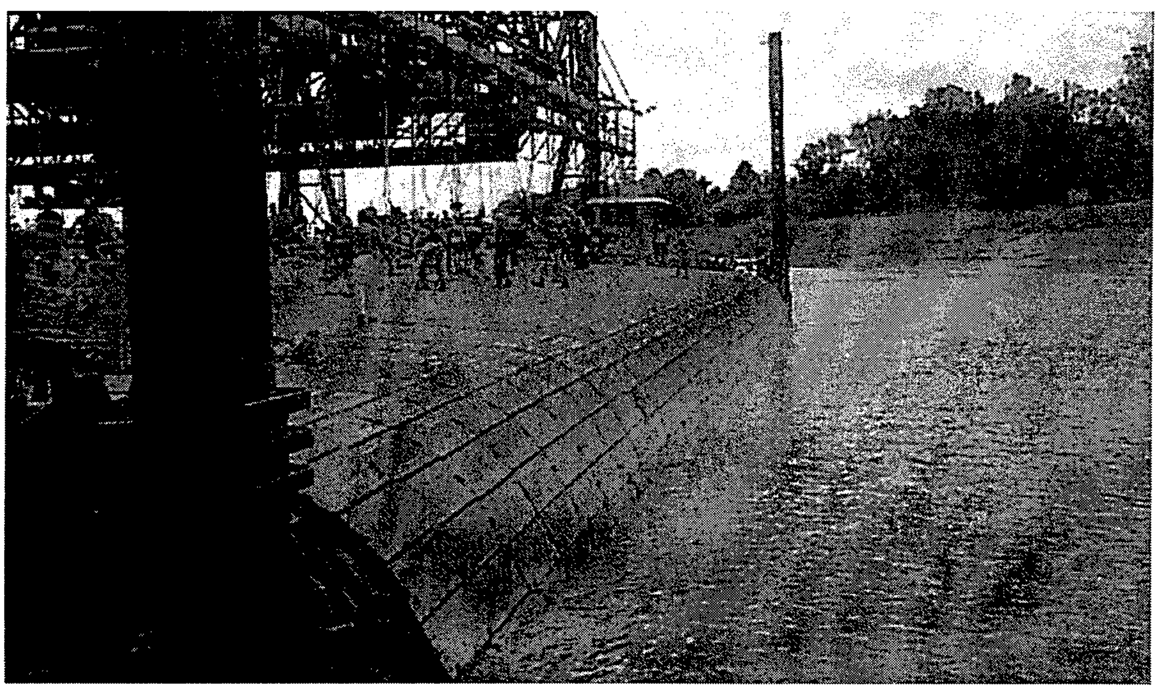

Figure 22. Machinery for laying articulated mattresses on riverbanks

Underwater Concrete

When constructing locks and dams in the wet, inevitably some of the construction will include cast-in-place concrete to be placed underwater. There are a number of methods for placing concrete underwater, as well as a variety of construction methods that can be used to improve placing and performance of concrete placed underwater. These methods will be discussed below.

Methods for placing underwater concrete

The numerous methods for placing concrete underwater include tremie, pumping, bottom-dump buckets, preplaced-aggregate concrete, slip-formed concrete, and precast concrete. Precast concrete will be discussed in a subsequent section ("Precast Elements"). The primary types of placement that are discussed in this section are tremie and preplaced-aggregate concrete.

Tremie concrete is the most popular method for placing underwater concrete, except possibly for precast concrete. The tremie system is essentially made up of a rigid tube (the tremie) with a hopper at the top of the pipe, through which the concrete is placed and then travels through the pipe under its own weight. Once the tremie placement has started, the bottom of the tremie pipe should remain embedded in the concrete already placed, as shown in Figure 23 (Yao, Berner, and Gerwick 1999).

Preplaced-aggregate concrete is another method for placing concrete underwater. This type of placement is usually employed when preparing a base. For this type of placement, a network of grout tubes is placed on a prepared surface and then covered with stone or riprap. Once the rock has been put into place, grout can be pumped in through the grout tubes and the grout will fill the voids in the layer of rock.

Self-leveling, self-compacting, and antiwashout concrete

Placing concrete underwater does not easily lend itself to vibration that is often used in conventional concrete placement to consolidate and level the concrete. While vibrators for use underwater are available, it is not the preferred method for consolidating and leveling concrete underwater since it could disturb the concrete and would be difficult to perform in a river environment. Another option that has been considered in the past is to drag a heavy beam across the surface of the wet concrete. Again, this is not desirable due to the disturbance it would create.

Another option is to develop a concrete mixture that is self leveling and self compacting. Significant investigation of self-leveling and self-compacting concretes has been performed in the last 10 years. Okamura and Kazumasa (1996) discuss development of a self-compacting concrete that is highly viscous and highly deformable by making use of superplasticizers. Use of this concrete is

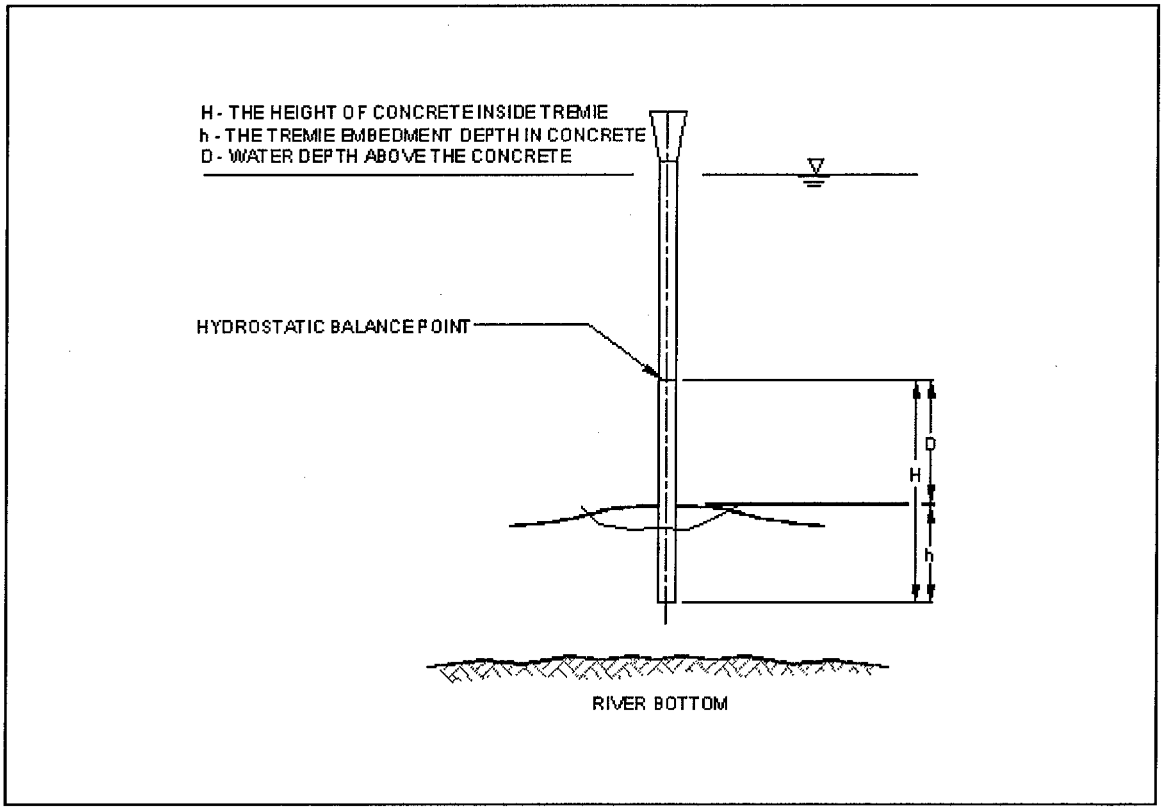

Figure 23. Use of tremie pipe in placing tremie concrete

also noted in the construction of the Akashi-Kaikyo Bridge in Japan (Tanaka et al. 1993). Self-leveling and self-compacting concrete (termed super-workable) was investigated extensively and shown to provide adequate ability to flow around reinforcement and embedded structural elements while providing some degree of self leveling (approximately 0.1-m variance over 13-m distance).

Placing concrete underwater also presents the concern over erosion and washing of the concrete as it is placed. Admixtures are available that can be added to concrete for underwater placement that will increase its viscosity and improve the concrete's resistance to segregation due to washing action of water (Nagataki 1993). In addition, use of an antiwashout admixture is often beneficial in achieving self-compaction and self-leveling properties, as discussed in the previous paragraph.

Further information on placement of underwater concrete is available not only in the references listed in the above discussion, paragraphs, but also in Technical Report INP-SL-1 (Yao, Berner, and Gerwick 1999).

Grouting voids under precast structures

In many of the float-in or lift-in structures that will be built for navigation projects, there will be a need to fill voids under the structures with grout to provide a uniform bearing for the monolith. For this type of concrete work, the properties should typically include low heat generation, homogeneous concrete, the capability for completeness of filling, cohesiveness, low bleed, and long-term stability (Gerwick 1999). In addition, the strength and modulus of elasticity will typically be low, since a high-modulus grout could create hard points on the base that would result in concentrated loads.

A key aspect of filling voids under precast structures using grout is to ensure that trapped water under the base is allowed to escape. This can be accomplished by using sloped surfaces that lead to vent holes, or by using inverted channels along the base. In some cases, bond to the surface of the precast unit may be important; thus, flushing of all water as well as any laitance that may form at the top of the placement is critical in this case. Use of antiwashout concrete will often be beneficial, particularly in a river environment where water is flowing.

Precast Elements

Many of the proposed plans for innovative navigation projects include the use of precast elements for float-in and lift-in construction. With the advent of such uses of precast concrete structures, there are some considerations that must be taken into account when preparing to design and construct such structures. The paragraphs below discuss some of the concerns and practical ideas associated with use of precast concrete.

Construction

Float-in and lift-in elements will often be constructed in an existing graving dock or in a graving yard constructed by the contractor along the river. The first consideration that needs to be made when selecting the location for fabricating the precast structures is the height of the units and possible interference with bridges or powerlines that may cross the river between the graving location and the project. Casting of the precast elements will normally occur on a layer of crushed rock covered by corrugated steel, plastic or polyethylene-covered plywood sheets, or on a hard-finished concrete slab. (Usually, a bond breaker will be applied to the hard-finished concrete prior to placing the concrete for the precast element.) After the initial placement, the construction of the precast element will be carried out in a similar manner to cast-in-place construction. This will include joint preparation at any cold joints accommodating reinforcement from one lift to the next.

Once a precast element is complete, it will be launched from the graving site into the river. For a specially constructed graving yard, the area where the precasting took place will be flooded, and the floating precast element can be

pulled out and towed to the site. Of course, if the precast elements are open ended, these openings will need bulkheads to maintain buoyancy. For an existing casting yard, the unit can slide down a set of skids into the river to be towed to the site.

Another option for casting of precast elements is to cast the structure on a large barge or on several barges. When using this approach, care must be exercised in ensuring that the barges can withstand the external hydrostatic pressure during maximum submergence. For two or more barges connected together, design of the connections must be performed to ensure that the bending moments, axial forces, and shears can be maintained by the connection. Use of barges may be cost effective, particularly if the other option is to construct a graving site (Gerwick 1999).

Placement

Placing the precast elements at the construction site requires that both horizontal and vertical control be maintained. Vertical control is accomplished through proper ballasting of the unit. Horizontal control can be achieved through the use of moorings to preplaced anchors along with winches. A frame on top of a unit that extends above the water can be beneficial in helping to align a structure monolith with the previously placed monolith. If it is difficult to get exact placement when lowering the structure into place, an alternative is to maintain some of the buoyancy in the structure, after getting it within about 6 in. of its final location, and then to drag it into place.

If the units are on barges, the barges must be ballasted to a point from which the precast elements may be launched. Ballasting of the barge should be performed in stages to verify the stability of barge/precast buoyancy. Once the concrete unit is sufficiently immersed to float it off the barge, the barge has little or no stability. To assist in this process, it is beneficial to launch the precast element in an area where the riverbed is level and the barge will have to sink only a few feet. After the precast unit is towed clear of the barge, the barge can be pumped to remove ballast and raise it off the river floor (Gerwick 1999).

Float-in with auxiliary buoyancy

Auxiliary buoyancy may sometimes be needed to achieve the needed draft for transportation of the float-in structure. It may also be needed to provide additional stability during transport or installation of monolith. Typically, buoyancy will be achieved by using steel tanks, although concrete tanks have also been used in the past. Other supplemental buoyancy approaches include the use of foam blocks and inflatable rubber bags (Gerwick 1999).

When using buoyancy tanks, the design must ensure that tanks are able to withstand the hydrostatic pressure, including becoming overly submerged due to an accident. In addition, the tanks must be attached to the structure in such a way that does not create an overstress condition. The tanks may be connected with a

hinge or rigid connection. Fatigue should also be considered in the design of the connections. In addition, cylindrical tanks may be constructed so as to be in a vertical position during the sinking operation.

Culverts/Pipes

Construction of culverts underwater can be accomplished by a variety of means. Many of the processes that can be used for placing culverts can be derived from placement of pipes in the offshore industry and from experience gained in the placement of underwater tunnels. The following paragraphs outline some of the processes that have been used for the placement of pipes and tunnels underwater.

One method used by the offshore industry is pipe-laying barges. A pipe-laying barge performs pipe placement on the ocean floor by assembling sections of the pipe on a barge and then lowering the pipe through the ocean to the ocean floor (Figure 24).

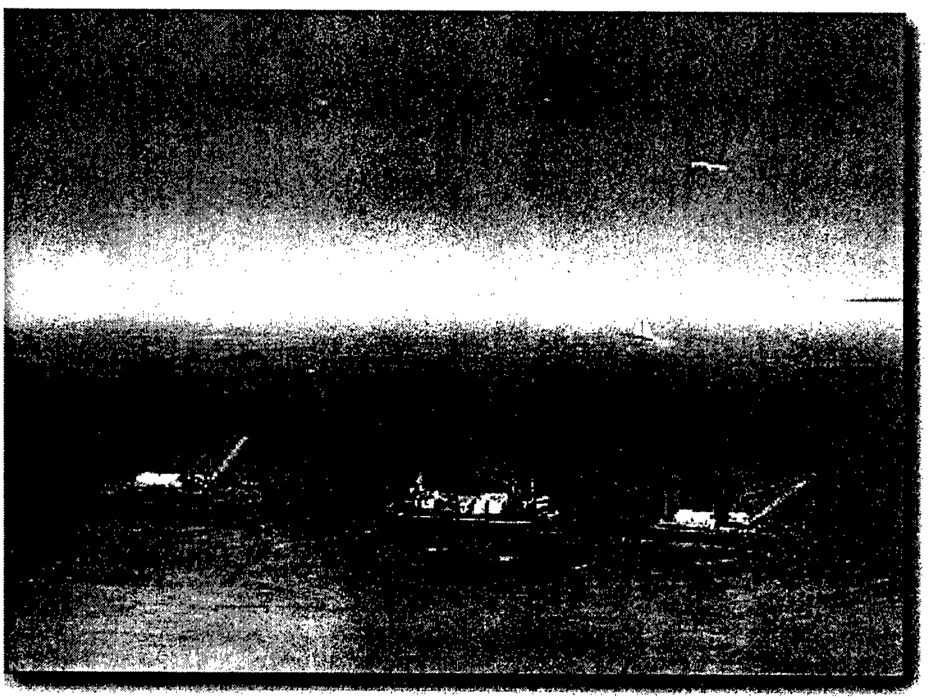

Figure 24. Pipe-laying barge

While the barges used for laying pipes in the ocean are very specialized vessels, the concept could be used for placing culverts in the chamber of a lock. Whether the culvert is to be round or rectangular, the mechanisms for making connections on the barge(s) could be easily assembled. The assembled portions could then be laid into the chamber in a progressive manner. Bending stresses in the culvert during the laying process could be reduced by attaching temporary

buoyancy to the culvert or by providing a stringer that supports the pipe after it comes off of the barge, as is done in the offshore industry. A method called the "S curve" technique uses a series of buoys that keep the pipe floating. Through careful release of the buoys, the pipe is allowed to settle to the floor. A variation on this is to place a fixture at each end of the pipe, to which a line may be attached to lead to the pipe, and then winch the pipe to the floor, releasing the buoys after the pipe has reached the bottom (Walther 1979).

Another method of pipe placement would be to place pipes underwater in sections and then join them underwater. Construction of tunnels often occurs in this manner. The tunnel sections are large, and will be floated into position and then lowered next to the previous section. As with the precast, float-in units, auxiliary buoyancy may be used to reduce the load needed for the cranes performing the lowering operation. To ensure proper positioning, tapered guides and tensioned guidelines may be used (Gerwick 1969). If the ends of the pipes are provided with bulkheads, the pressure of the water can be used to complete the final seating of one pipe section to the other, if both sections are dewatered. Often, a joint gasket will be placed at this final seating location. This approach was used in the construction of the Ted Williams Tunnel, which is part of Boston's Central Artery/Tunnel Project (Lancellotti and Grantz 1997). A similar approach is often used when assembling pipelines for coastal sewage treatment facilities (Ganas 1989).

Temporary Enclosures

Temporary enclosures that can be easily placed and removed in relatively short periods of time can be cost-effective ways of achieving construction of various components. Temporary enclosures provide the opportunity to construct portions of a project in the dry, without requiring the construction of a large structure such as a cofferdam. Some temporary enclosures that have been used during construction of in-the-wet projects are discussed below.

Underwater pressurized cells

One method that can be used to perform work underwater is to employ an underwater pressurized cell. The cell becomes an open-bottom dive bell in which the pressure inside the cell is equal to the pressure of the water at the bottom of the cell, which keeps water out of the cell. This allows a diver to essentially work in the dry and with good visibility, except for a small amount of water at the bottom. Dead weights can be attached to the cell to keep it from floating and typically will be small enough to be lifted by a single crane and moved from one location to another (Rail and Haynes 1991).

Alternative cofferdam structures

While sheet-pile cellular cofferdams have been used extensively for the construction of major components of locks and dams, other types of cofferdams or caissons have also been used or proposed.

A mobile cofferdam system was considered in the Phase I Feasibility Study (USACE 1993) by the task force on Design and Construction Innovations for Locks and Dams. The concept is to build a unit large enough that one or two monoliths of a lock or dam could be built within the unit. The unit would have ballast cells that would allow it to be easily moved from one location to another. The mobile cofferdam could be floated to the proper location, sunk, and a layer of tremie concrete placed at the bottom of the mobile cofferdam to provide a seal. A concrete monolith could then be constructed within the cofferdam using cast-in-place concrete and/or slip form technology. Once the monolith within the mobile cofferdam is completed, the cofferdam could be floated to the next location and the process could begin again.

Another type of cofferdam that has been used is the fabricated steel caisson. Caissons are open sided, open ended, or both. They are designed to rest on underwater concrete structures, or to fit and seal against near-vertical concrete surfaces. Caissons have been used to repair miter gate sills, to construct recesses and appurtenances on lock wall faces and concrete dams, and in general, to provide dry access to underwater portions of structures.

Cofferdams of various descriptions have been proposed, and some are being built to serve as a part of the permanent lock structure. This application, however, is not a part of this study.

Monitoring Placement

Once the method for placing items has been determined, the question arises as to how to monitor the placement so that the components of the structure are placed within acceptable tolerances. This is a particularly difficult task when the items that need to be monitored are in a marine environment and becomes even more of a challenge when these items will be totally immersed in murky water. Another issue that increases the difficulty of monitoring structure placement in rivers is the desire to reduce or eliminate the use of divers. Some possibilities for monitoring placement of structures in the wet are presented below.

If portions of the structure being placed will project above the water surface, traditional methods of surveying may still be used. Several points on the exposed portion of the structure must be selected, and performing a survey of these points can ensure that the structure is placed at the correct elevation and is positioned properly in both the lateral and longitudinal directions.

Alternatively, global positioning system (GPS) receivers can be placed on exposed points of a structure to achieve the same objective as was demonstrated

during the placement of a protection cell at Lock 24 on the Mississippi River (Mann and Fehl 1998). GPS has now achieved accuracy levels that will allow measurement to be within 1 in. or less.

Conventional surveying methods and GPS may still be used even if a component will be completely submerged. This can be accomplished by attaching range poles to the component to be submerged so that the tops of the poles will be above the water surface. Care must be used when applying this approach. The poles should be designed structurally so that they will not deflect significantly in flow and will not be damaged by floating debris or by starting and stopping operations during placement.

Consideration should also be given to the use of remotely operated vehicles (ROV). Cameras on ROVs are, of course, of little or no value in the muddy environment of the Mississippi and Ohio Rivers, but with advances in the technology, ROVs can now be equipped with a number of sensors and positioning devices (Gittleman 1993). Acoustic position-finding devices can be used to accurately determine the relative position of a structure in a three-dimensional space (Gerwick 1999).

For components that are completely submerged, side-scan sonar can be used. Side-scan sonar produces a sonar image that looks very much like a picture of the bottom of the river. While it may not be extremely beneficial during the actual moving and placing of structures, this technique could be valuable for use after placement, to ensure that the component is near the proper location. It could also be used to confirm the condition of the foundation after preparation, and periodically thereafter. While side-scan sonar has not achieved an accuracy level that allows it to be used as a final determination of the condition of the foundation or underwater components, it may be valuable for assessing conditions prior to sending divers down to obtain more accurate measurements.

Applications of Innovative Construction

The above paragraphs describe a number of different methods and techniques to be used in performing construction in the wet. How can these methods and techniques be used for the construction of lock floor slabs and culverts? To answer this question, the various plans discussed in Chapter 2 will be used as a basis to describe how the techniques and methods described above can be used to accomplish the construction of these plans.

Plan 1

Recall that Plan 1 is a simple plan developed from the UMR-IWS Navigation Study in which an existing 600-ft lock is to be extended to a length of 1,200 ft (see Figures 1-3). This plan will use the existing filling and emptying system of the 600-ft lock, so no requirements for a culvert must be considered. However, there will be a paving slab.

The paving slab is structurally independent of the walls and essentially provides a surface to define the floor elevation of the lock. The paving slab may also serve as a work platform when the lock is dewatered for maintenance. Several methods could be used to construct the paving slab and will be discussed below. However, before placing of the paving slab, an underlying bedding/filter layer must be placed. This layer is needed to allow seepage through the foundation and up through the paving to relieve uplift pressures that may act on the bottom of the paving slab.

Prior to placement of the bedding material, unsuitable sediments will need to be removed using either a clamshell or a dredge. Foundation conditions will vary, but a geotextile membrane may be required under the bedding material. The bedding may then be placed to an elevation that is near the desired base elevation of the paving. A heavy beam may be used to drag along the top of the bedding material to achieve the final grading. A vibratory plate compactor may be necessary to consolidate loose bedding material prior to placing the paving.

After placing the aggregate filler, a couple of methods could be used for placing the paving slab. One approach would be to use tremie concrete. For this case, a self-leveling, self-compacting, antiwashout tremie concrete could be used to provide a competent, evenly placed slab. Some pipes may need to be embedded in the tremie concrete to provide relief from uplift. As mentioned in Chapter 2, individual, precast paving blocks could be used. Constructed in a precasting yard, these blocks would ensure quality control of the concrete; weep holes could be cast as needed. The size of the paving blocks could be limited to the lifting capacity of readily available cranes. An alternative to placing the paving blocks individually would be to develop a method for building an articulated concrete mat on a barge (similar to the mat sinking units used for bank stabilization on the lower Mississippi River) and to lay this articulated mat across the bottom of the lock chamber.

Plan 2

Plan 2 consists of a structural floor system that spans between the lock walls. It is also the extension of a 600-ft lock and was taken from the UMR-IWS Study. An alternative to the wall design was proposed, as well. Illustrations of the plan are shown as Figures 4 and 5, with a section of the alternative plan shown as Figure 6.

As in Plan 1, the foundation will first need to be prepared. Again, excavation by clamshell or dredge must be performed. If the area being prepared has no silt, a leveled layer of scour stone may be all that is required prior to placing the floor system. If siltation is a problem, excavation of the silt and placement of geotextile membrane and bedding may be required prior to placing the floor system.

Weep holes are not necessary in this plan since the lock floor slab is a structural member and will be designed to resist uplift.

The slab is intended to be a precast, prestressed concrete member. The structural floor system will likely be floated into position and sunk in place. Construction of such a component could be performed in a casting yard near an inland waterway and transported to the site by barge. If the barge is submersible, it could be sunk at the project site to allow the slab to be floated into position. Otherwise, one or more cranes would lift the floor slabs off of the barge and into the water. Design of the precast slab would also require ports that would allow the placement of grout beneath the floor slabs.

After positioning the slab, ballasting of the slab unit could begin. Careful computations are needed to perform such an operation. During the ballasting operation, the slab unit would be attached to a crane that could keep the unit stabilized to some degree when its buoyant weight begins to become negative.

Once the slab is in place, the connection to the wall must be made. This may require a diver. However, since ROVs have advanced significantly in the last decade, it may be possible to equip an ROV with a manipulator that could perform the connection.

Once the connection is complete, placement of the grout under the floor slab can be performed. Design of the slab should include ports for placing the grout and ports that allow water trapped under the unit to escape. If possible, the bottom of the unit should slope toward the ports that allow water to escape. The grout should have properties as described earlier. Likewise, the concrete used to fill the voids in the slab unit should be flowable and have a low heat of hydration. Design should be performed in these areas, as well, to allow trapped water to escape.

The only difference that occurs when considering the alternative plan is that consideration for placing the culvert is necessary. Based on the cross-sectional shape shown in Figure 6, it would appear that placing the culvert in segments might be the most likely alternative. It is expected that these segments could be lifted into place. The placing of these segments will not occur end to end, as discussed earlier. Rather, the monolith to which the culverts are being affixed should have tapered sections to receive the culvert segments. Some attention also needs to be given to joint sealing between the culvert segments and the monolith. If concern about damage or movement to the culverts exists, tremie concrete could be placed over the segments.

Plan 3

Plan 3 is another plan taken from the UMR-IWS Study and is intended to be used in conjunction with a miter gate bay that exists at several locks on the upper Mississippi River. This plan combines a structural system with a nonstructural paving. Figures 7 and 8 show cross sections taken at two locations along the length of the lock. Figure 7 is taken at a location where paving blocks are shown to be covering the lock floor; Figure 8 is taken at a location where a structural beam spans the lock chamber.

Initial foundation preparation will require removing foundation material to a given elevation. This excavation will be to the same elevation for both the area where the paving will be placed and the area where the beams will be located. After excavation, 1 ft of leveling stone will need to be placed throughout the chamber. A remote-controlled underwater bulldozer may be used to level the stone.

After the leveling stone has been placed and leveled, the concrete beams would be placed. Accurate placement of the beam will be important for both the transverse and longitudinal directions because it will need to be integrated with the structural system of the wall. The precast structural floor beams will be several feet shorter than the chamber width. The areas between the ends of the beams and the walls will then need to be cast in place. An enclosed form could possibly be constructed to fit over the beam and against the wall, and then concrete could be tremied into this area.

Once the beams are in place, the paving area can be completed. Prior to placement of the paving system, the graded foundation would need to be placed. The plan shows the placement of a geotextile fascine mattress, 2 ft of filter sand, and 2 ft of drainage gravel. As in Plan 1, the geotextile mattress could be placed in a similar manner as when geotextile mattresses are placed prior to riprap placement. Placement of the sand and gravel would follow, with leveling accomplished using the screeding method, which employs a heavy beam (as discussed in Plan 1 as well).

Placement of paving blocks is the final step and can be accomplished using the approaches discussed in Plan 1. This would include placement of blocks individually so that the size would need to be limited to a practical size for lifting, placement of the blocks as a mat (similar to the mats sunk in the Mississippi River), or use of tremie concrete. As in Plan 1, regardless of the paving system used, weep holes will need to be included to reduce uplift pressures.

Plan 4

Plan 4 consists of a culvert system that is integrated in the floor system. A cross section of the lock chamber for Plan 4 is shown as Figure 10. A detailed section cut of the floor/culvert system is shown in Figure 11. As with the previous plans, Plan 4 was taken from the UMR-IWS Study and was developed as part of a plan to build a new lock at an existing dam. A plan view of the entire lock is shown in Figure 12, with a plan of the miter gate monolith shown in Figure 13 and the section elevation in Figure 14. The filling and emptying process is described in Chapter 2.

Excavation for the precast unit can be accomplished as discussed in previous plans. The plan as described in the UMR-IWS Study calls for placement of leveling stone prior to placement of the precast unit. Placement of leveling stone could be accomplished as described for Plan 3. An alternative might be to place flat jacks on the excavated foundation and place the precast unit on top of the flat

jacks. The flat jacks can then be used to raise the precast unit to the required elevation; then, tremie concrete can be placed underneath the unit using a grout, as described earlier.

The precast concrete shells can be fabricated offsite and then floated or lifted into place. Once the precast unit has been properly positioned, the piling can be driven through knockouts in the bottom of the precast unit using underwater pile-driving equipment. Placement of the grout under the unit should not be done until after the piles have been driven. This should be followed by tremie placement of concrete around the pipes used to form the culverts. Smaller pipes will need to be attached to the larger conduit pipes to form outlets. Attachments to these smaller pipes, to divert flow horizontally, will also be required.

The current plan shows that the connection of the floor/culvert system and the wall is accomplished by placing tremie concrete between the wall and the precast unit. Piles must first be driven in this area. Reinforcing (extending from the wall and the precast unit) is also needed to provide a proper tension connection. Reinforcing extending from the lock wall concrete shell will be placed by a diver and spliced with reinforcing extending from the precast lock floor system. The lock wall concrete shell will be filled with tremie concrete, thereby encasing foundation piles. Tremie concrete will be placed in forms to complete the tie between the lock wall and the precast lock floor system. The possibility exists that interlocking connections might be made between the lock walls and the precast lock floor system. This alternative can be explored during the design stage.

Plan 5

Another plan taken from the UMR-IWS Study is shown in Figures 15 and 16. In this plan, the culverts extend through the sill and down the length of the chamber. This is another plan that was laid out with the intention of placing a 1,200-ft lock through an existing dam and was part of the UMR-IWS Study. The inlet and outlet system to be used is the same system described for Plan 4 in Chapter 2. The culvert and floor system is to be constructed with sheet-pile cellular lock walls. The lock floor around the culvert will consist of riprap material. Since the culvert does not extend the entire length of the lock, the portion of the lock where there is no culvert will be paved.

As will all of the previous plans, excavation will need to be performed in the chamber and can be accomplished as described previously. Once the excavation has been completed, the foundation capacity of the area beneath the culverts needs to be increased. Piles driven with underwater pile-drivers can be used to provide the support needed. Another option would be to use minipiles (described earlier) instead of conventional piles. Minipiles may be more cost effective than full-size piles. Still another option is to use soil mixing. As discussed earlier, soil mixing uses an auger to drill into the soil and mixes a cement/water grout with the soil to increase its stiffness. Soil mixing can improve the compressive strength of soil up to 200 psi. If stiffness beyond this capacity is needed, soldier piles can be included in the soil-mixing process.

To prevent flotation, connections to pile-supported culverts need to be capable of transmitting tension. This will be accomplished by driving Z-piles alongside the culverts. Voids between the culverts and the Z-piles will be filled with tremie concrete. Voids between the steel sheet-pile cellular structures and the culverts will likewise be filled with tremie concrete.

After placement of the leveling stone is complete, the culverts may be floated or lifted into place. The culvert sections will be precast offsite. A connection of the culvert sections underwater will need to be achieved. This will require connection details that allow one section to be guided to the connection of the previously placed section. In placing culvert sections such as these, methods for connecting tunnel sections may be used as described above. In addition, information from the related INP work unit "Modular Connections and Seals for Precast Segments" can be used to better evaluate these types of connections. It should be noted that this plan, as shown, has port openings directed vertically. The plan will need to be modified to deflect vertical flow through the ports horizontally into the lock chamber.

Placement of bedding material and riprap around culvert areas will complete the culvert installation. Some care will need to be exercised in placing riprap so that the culvert sections are not damaged in the process. Also, to ensure that no the riprap remains on top of the culvert sections, side-scan sonar may be used to view the completed lock floor and culverts. The downstream portion of the lock chamber will be paved by the most cost effective of the various plans described previously.

Plan 6

Plan 6 was taken from the Prototype Alternatives Study done for the Ohio River Main Stem. A plan view of the 1,200-ft lock to be constructed is shown as Figure 17. The plan is to be constructed in the dry, except for the portion of the culverts crossing the downstream approaches of the two existing locks and the new lock. As opposed to the first five plans, this plan is to be constructed on a rock foundation.

The foundation will first need to be prepared to provide the proper support for the pipe. Small variations in foundation grade may be corrected by using a grinding wheel similar to those used to excise asphalt from roadways. Close monitoring of the grinding operation will be required to ensure a uniform grade.

After the rock has been leveled, the rock surface must be further prepared to ensure that the pipe has adequate bearing. Leveling stone may be used, and can be placed and leveled as discussed for Plan 3. Use of a series of flat jacks could also be a means to get the pipe sections at the proper elevation. Grout fill can be used to ensure uniform support for the pipe.

This plan provides a good opportunity to use procedures applied in tunneling projects to join pieces. Tunneling projects would provide a number of methods for making connections, as well as the procedures for accomplishing the actual

joining of the pipe sections. As mentioned previously, another INP work unit is investigating connections of modular units and may provide pertinent information with regard to this issue. Connections at the walls would need to be considered, and these connections should be included in the design process of the walls.

Plan 7

Plan 7 is also taken from the Ohio River Main Stem—Prototype Alternatives Study. This plan, shown in Figure 18, is for the extension of a 600-ft auxiliary lock. The inlet of the culvert is upstream of the upstream guide wall and extends along the length of the lock until it transitions into the chamber. This portion of the project will be built in the dry. The portion of the system contained within the chamber may be constructed in the wet.

This project is also to be founded on rock, so there is nothing to consider with regard to the lock chamber floor except the possibility of some grinding of the rock to provide a level floor. However, the culvert and laterals are to be embedded in the rock. Grinding of the rock was described in the previous section. A grinder that moves horizontally along the rock surface would be satisfactory for leveling the rock foundation; however, excavating trenches in this manner may be difficult. Using a drill with a vertical shaft may the most feasible approach for digging the trenches needed for embedding the culverts and laterals. Blasting may also be an alternative, although it would not appear to be a desirable method due to the proximity of the existing main lock chamber.

As with other culvert sections that have been discussed in regard to other plans, placement using methods for tunnel construction would appear to be the best alternative. Design of the wall where the culvert enters would need to accommodate this approach, as well as the miter gate sill where the culverts exit. A prepared base will also be needed for the culverts, as well as laterals to provide structural support. A prepared stone base or flat jacks and tremie concrete, as described for Plan 8, could also be used in this plan.

5 Summary and Recommendations

Summary

A variety of plans were presented, with the intent of providing an array of construction plans that might be undertaken for in-the-wet construction. While these plans may not match every plan that is used, they provide the basis for investigating a number of different construction technologies that can be used for in-the-wet construction and were an aid in trying to define design parameters that might require adjustments due to construction being performed underwater instead of in the dry. In-the-wet construction techniques employed on other projects were reviewed for application with respect to the plans presented, and suggestions were offered as to how these technologies might be used.

There was also an extensive presentation of the design parameters that would be considered for each of the plans. From the discussions in Chapter 3 on design parameters, a list of recommendations was developed which addresses where deficiencies might exist for the type of innovative lock floor slabs and culverts that may be constructed. These recommendations are presented in the next section.

Recommendations

As a result of the discussion presented in Chapter 3, it is recommended that further investigation be undertaken on the design parameters identified below.

Flotation factors of safety

ETL 1110-2-307 gives the required factors of safety against flotation. Uplift resulting from seepage forces may not be precisely determined by analyses with unknown parameters. Loading conditions for which flotation factors of safety will be determined are for the usual, unusual, and extreme loading conditions. The corresponding required factors of safety are 1.5, 1.3, and 1.1, respectively.

Sensitivity studies are suggested as a means of determining the probability that these factors of safety are adequate for structures subjected to uplift due to seepage forces. Predictions of the coefficients of permeability of soils, the effectiveness of seepage cutoffs, the lengths of seepage paths, and the effectiveness of pressure relief measures are less accurate than uplift calculations using static hydraulic pressures. To account for these uncertainties, studies may be needed to determine new factors of safety that will provide the same degree of safety as those prescribed by ETL 1110-2-307 for loading conditions other than underseepage. Values of the coefficients of permeability of soils are based on grain size, pumping tests, and laboratory tests on undisturbed and remolded soil samples. Seepage analyses are made with flow nets and by various computer analyses that include grid relaxation techniques. The effects of pumped wells, including the rates of pumping, can be evaluated by two- and three-dimensional computer analyses. Stratification of foundations can be evaluated using transformed sections. Anisotropic soils resulting from stratification result in differing coefficients of permeability for vertical and horizontal seepage. Coefficients of permeability that vary due to anisotropic conditions can be accounted for using transformed sections and computer analyses.

Sensitivity analyses can be made with computers using the above variables and probable distributions of parameters. It is highly probable that higher factors of safety would be required to provide the same level of confidence as that provided by the above factors of safety applied using reasonable predictable uplift values derived from static heads. The above may also be applied to the critical exit gradient, although the values used for embankment dams may be adequate. The results of these studies should be documented for future guidance.

Capacity of piles driven in water

Guidance may need to be developed for determining the capacity of piles driven in water if field tests, library research, or laboratory tests indicate there is a need for guidance other than that in EM 1110-2-2906.

Design parameters for high-performance concrete

Laboratory tests may be needed to determine if design parameters given ACI 318 and EM 1110-2-2104 are appropriate for high-performance concrete. Values for shear strength, bond strength, and reinforcing development and splice length are of primary concern.

Sheet-pile cutoff walls

Sheet-pile cutoff walls driven beneath pile-supported concrete gravity structures are crucial to the control of seepage. A condition referred to as "roofing," i.e., voids caused by the settlement of soil foundations beneath structures founded on piles, places great reliance on cutoff walls. The depths to which sheet-pile walls can be driven without damage to interlocks, and the depth

to which piling needs to be driven for seepage control, should be evaluated. Some of the conceptual plans call for piling to be driven 32 to 40 ft. These depths would be difficult to achieve under any but the most favorable conditions.

Seepage control

The applicability of factors of safety given in EM 1110-2-1901 needs to be evaluated for use with navigation structures. As many navigation locks have foundations with fine sands and silts, a potential for foundation piping exists if factors of safety against critical escape gradient are not adequate.

References

American Association of State Highway and Transportation Officials. (1996). *Standard specifications for highway bridges*, 16 ed. Washington, DC.

American Concrete Institute. (1999). "Building code requirements for structural concrete," ACI 318R-99, Farmington Hills, MI.

American Society for Testing and Materials. (2001). "Standard specification for deformed and plain billet-steel bars for concrete reinforcement," A615. West Conshohocken, PA.

Craeger, W. P., Justin, J. D., and Hinds, J. (1945). *Engineering for dams*. John Wiley and Sons, New York.

Duncan, C. R. (1972). "Deep underwater rock excavation for bridge pier," *Highway Focus* 4(3). U.S. Department of Transportation, Federal Highway Administration, Washington, DC.

Ganas, M. J. (1989). "Assembling underwater concrete pipelines," *Concrete International: Design and Construction* 11(9).

Gerwick, B. C., Jr. (1969). "Construction of large concrete ocean structures," *Journal of the Construction Division* (ASCE) 97.

__________. (1999). *Construction of marine and offshore structures*, 2nd ed. CRC Press, Boca Raton, FL.

Gittleman, M. M. (1993). "Undersea applications of dexterous robotics." *Workshop on space operations applications and research (SOAR '93)*, Houston, TX.

Headquarters, Department of the Army. Washington, DC. Guidance documents—Civil works. http://www.usace.army.mil/publications/.

EM (Engineer Manual) 1110-2-1901, Seepage analysis and control for dams.

EM 1110-2-2104, Strength design for reinforced-concrete hydraulic structures.

Headquarters, Department of the Army. Washington, DC. Guidance documents—Civil works. http://www.usace.army.mil/publications/.

EM 1110-2-2906, Design of pile foundations.

ETL (Engineer Technical Letter) 1110-2-307, Flotation stability criteria for concrete hydraulic structures.

Lancellotti, A. R., and Grantz, W. C. (1997). "From concept to completion: The Ted Williams Tunnel." *Proceedings, Immersed Tunnel Techniques 2, international conference*, C. Ford, ed. Cork, Ireland.

Mann, R. G. (1998). "Monitoring steel cell placement using differential GPS at Lock and Dam 24, Mississippi River." http://crunch.tec.army.mil/information/ publications/Lock_Dam24/cell.htm.

McCammon, N. R., and Fitzell, T. P. (1991). "Foundations for a concrete caisson wharf for ocean-going ships on the west coast of Canada." *44th Canadian geotechnical conference*. Golder Associates, Ltd., Vancouver, B.C., Canada.

Nagataki, S. (1993). "Use of antiwashout underwater concrete for marine structures," Technical Report No. 47, Department of Civil Engineering, Tokyo Institute of Technology, Tokyo, Japan.

Okamura, H., and Kazumasa, O. (1996). "Self-compacting high performance concrete." *Structural Engineering International*.

Pearlman, S. L., and Himick, D. E. (1993). "Anchored excavation support using SMV (soil mixed wall)." *18th annual member's conference*. Deep Foundation Institute, Pittsburgh, PA.

Post-Tensioning Institute. (1996). *Recommendations for prestressed rock and soil anchors*, 3rd ed. Phoenix, AZ.

Rail, R. D., and Haynes, H. H. (1991). "Underwater stilling basin repair techniques using precast or prefabricated elements," Technical Report REMR-CS-38, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Tanaka, K., Sato, K., Watanabe, S., Arima, I., and Suenaga, K. (1993). "Development and utilization of high performance concrete for the construction of the Akashi-Kaikyo Bridge." *High performance concrete in severe environments*, Special Publication 140. American Concrete Institute, Farmington Hills, MI.

U.S. Army Corps of Engineers. (1993). "Design and construction innovations for locks and dams; Phase I, feasibility," Washington, DC.

U.S. Army Corps of Engineers. (1997). "Second generation of Innovations for Navigation Projects (INP) Research and Development (R&D) Program, Workshop," Washington, DC.

__________. (1998). "Ohio River Main Stem study; Prototype alternatives."

U.S. Army Engineer Districts, Rock Island, St. Louis, and St. Paul. (1996). "Upper Mississippi River-Illinois Waterway System navigation study; Large-scale measures of reducing traffic congestion, conceptual lock designs." Rock Island, IL.

__________. (1999). "Upper Mississippi River-Illinois Waterway System navigation study; Interim revised lock extension design concept." Rock Island, IL.

Walther, E. (1979). "A look at sub-sea flow line installation," *Petroleum Engineer International.*

Yao, S. X., Berner, D. E., and Gerwick, B. C. (1999). "Assessment of underwater concrete technologies for in-the-wet construction of navigation structures," Technical Report INP-SL-1, U.S. Army Engineer Research and Development Center, Vicksburg, MS.

Bibliography

Allen, W. (1989). "Underwater inspections," *Concrete International* 11(9), 37-40.

American Institute of Steel Construction. (1989). *Manual of steel construction; Allowable stress design*, 9th ed. Chicago, IL.

American Welding Society. (2001). "Structural welding code," AWS D1.1. Miami, FL.

Antiwashout Underwater Concrete Research Committee. (1991). "Recommendations for design and construction of antiwashout underwater concrete," Japanese Society of Civil Engineers, Tokyo, Japan.

Billberg, P., Petersson, O., and Norberg, J. (1996). "New generation of superplasticisers." *Production methods and workability of concrete*, P.J.M. Bartos et al., ed. E&FN Spon, London, UK.

Castilla, P. (1984). "Fixity of members embedded in concrete," Technical Report M-339, U.S. Army Engineer Construction Engineering Research Laboratory, Champaign, IL.

Khayat, K. H., Sonebi, M., Yahia, A., and Skaggs, C. B. (1996). "Statistical models to predict flowability, washout resistance and strength of underwater concrete." *Production methods and workability of concrete*, P.J.M. Bartos et al., ed. E&FN Spon, London, UK.

Kuroda, Y., and Yamazaki, N. (1998). "Characteristics of lateral pressure of highly flowable concrete," *Proceedings, spring convention*. American Concrete Institute, Farmington Hills, MI.

Neeley, B. (1988). "Evaluation of concrete mixtures for use in underwater repairs," Technical Report REMR-CS-18, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

REPORT DOCUMENTATION PAGE

Form Approved
OMB No. 0704-0188

1. REPORT DATE *(DD-MM-YYYY)*
January 2003

2. REPORT TYPE
Final report

3. DATES COVERED *(From - To)*

4. TITLE AND SUBTITLE

Investigative Study for Underwater Construction of Lock Floors and Culverts

5a. CONTRACT NUMBER

5b. GRANT NUMBER

5c. PROGRAM ELEMENT NUMBER

6. AUTHOR(S)

Barry D. Fehl, Thurman W. Gaddie, Kevin Abraham

5d. PROJECT NUMBER

5e. TASK NUMBER

5f. WORK UNIT NUMBER
33147

7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)

GEI Consultants, Inc., 243 N Lindbergh Blvd, Suite 312, St. Louis, MO 63141-7851; Consultant, 151 Miami Lakes Drive, Milford, OH 45150-5809; U.S. Army Engineer Research and Development Center, Information Technology Laboratory, 3909 Halls Ferry Road, Vicksburg, MS 39810-6199

8. PERFORMING ORGANIZATION REPORT NUMBER

ERDC/ITL TR-03-1

9. SPONSORING / MONITORING AGENCY NAME(S) AND ADDRESS(ES)

U.S. Army Corps of Engineers, Washington, DC 20314-1000

10. SPONSOR/MONITOR'S ACRONYM(S)

11. SPONSOR/MONITOR'S REPORT NUMBER(S)

12. DISTRIBUTION / AVAILABILITY STATEMENT
Approved for public release; distribution is unlimited.

13. SUPPLEMENTARY NOTES

14. ABSTRACT

This report presents the results of a detailed investigation into the construction and design of lock floor slabs and culverts in an underwater condition (versus in the dry).

An extensive review process and discussions with Corps of Engineers Division and District representatives led to the selection of seven plans for inclusion in the intensive investigation. The plans selected were those that offered reasonable expectations for advantageous construction and represented the potential for application to a wide variety of construction technologies for in-the-wet construction. These plans were used to help define the design parameters that might need adjustment due to construction being performed underwater.

The innovative plans were evaluated against a set of criteria to identify the advantages and limitations of each plan and to determine its applicability to underwater construction. These criteria included issues related to maintenance, longevity, projected ease or difficulty of implementation (i.e. constructibility), risks to the operation of the lock, cost, environmental concerns, hydraulic efficiency, and construction time.

(Continued)

15. SUBJECT TERMS

Culverts	Locks
Design considerations	Piles
Floor slabs	Underwater construction

16. SECURITY CLASSIFICATION OF:			17. LIMITATION OF ABSTRACT	18. NUMBER OF PAGES	19a. NAME OF RESPONSIBLE PERSON
a. REPORT	b. ABSTRACT	c. THIS PAGE			19b. TELEPHONE NUMBER *(include area code)*
UNCLASSIFIED	UNCLASSIFIED			96	

14. ABTRACT. (Concluded)

The conclusions presented will permit designers to evaluate various options for constructing floor slabs and culverts with a better understanding of the benefits and the restrictions associated with a given approach. Further investigation is recommended for several design parameters: flotation factors of safety, capacity of piles driven in water, design parameters for high-performance concrete, sheet-pile cutoff walls, and seepage control.